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STREAM WATER QUALITY ASSESSMENT PROCEDURES MANUAL

Ontario
Environment
Environnement

STREAM WATER QUALITY ASSESSMENT

PROCEDURES MANUAL

MARCH, 1980

ONTARIO MINISTRY OF THE ENVIRONMENT
WATER RESOURCES BRANCH
WATER MODELLING SECTION
REGIONAL SERVICES

NOTE

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PROCEDURES MANUAL

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INTRODUCTION

The water resources of Ontario must meet many needs, and inevitably, some of these needs are in conflict. The ultimate disposal of waste waters in streams, rivers and lakes has been traditionally accepted as one of the legitimate uses of these waters. The fundamental concept governing the disposal of waste water and regulation of pollutants borne by it, has been to let treatment plants achieve part of the "clean-up" and let nature complete the job.

The intrinsic capacity of the receiving water body to assimilate constituents of wastewaters is commonly referred to as its "self-purification capacity". The acceptability of applying the principle of self-purification to wastewater disposal is contingent upon the maintenance of suitable water quality for downstream uses. These uses include public, agricultural and industrial water supply, recreation, aesthetic enjoyment and propagation of fish and wildlife. Water quality requirements for the various uses are clearly stated in the Ministry publication, "Water Management - Goals, Policies, Objectives and Implementation Procedures of the Ministry of the Environment" (November, 1978). The above publication states:

> "In essence a water body's dilution/assimilation capacity for wastes depends on waste characteristics and a host of physical, chemical and biological factors such as flow or volume of the water body and waste discharges, dispersion of the effluent, depth and width of the water body, type of substrate, algal growths, benthic deposits or organic sludges, etc.".

Clearly, an informed approach on a case by case basis is required to evaluate a water body's assimilative capacity and establish the effluent requirements to meet the Province's water quality objectives. To assist in achieving this, the Ministry has at its disposal a variety of assessment techniques.

The appropriate assessment technique to be used in a given situation depends on the characteristics of the pollutant under consideration and the state of the art of knowledge concerning its behaviour in the environment. In addition, constraints may be imposed on the selection of method by limitations on manpower, time and financial resources.

Where knowledge of the chemical behaviour of a pollutant is limited, and resource and time constraints prevent detailed study, a qualitative/descriptive assessment may be necessary. In this type of approach, inferences are drawn by describing cause-effect relationships based on a knowledge of the various factors and processes of importance. Where the processes involving a pollutant or water quality indicator are better understood, a quantitative assessment technique may be used. This generally involves the application of an appropriate mathematical model which simulates the processes of importance and predicts changes of water quality under different management strategies.

The purpose of this manual is to provide a guide to various aspects of water quality modelling most useful to personnel involved in the mangement of surface water quality problems. Initially, the necessary preliminary work, and the actual modelling of the DO-BOD process is covered. Future supplements to the manual will describe the application of the other models including the transport and dispersion of toxic substances. It is hoped that this manual makes the modelling approach more generally available to personnel involved in the assessment of water quality problems.

CHAPTER I

WATER-QUALITY - INFLUENCES AND ASSESSMENT

CHAPTER ONE

WATER QUALITY - INFLUENCES AND ASSESSMENT

1.1 Water Quality Assessment

This is a period of increasingly complex water quality problems, whether viewed from a scientific or a management viewpoint. The regional planner, in the face of this complexity is also confronted by increasingly definitive institutional controls on wastewater and receiving water quality. His immediate need is to obtain the appropriate technical means to evaluate the problems at hand within the time and resources available. This need is often met by quantitative or qualitative abstractions of the system being managed; these are collectively labelled as models.

The approach of managers in the field of water quality planning is basically a 4-step process involving (1) problem identification; (2) system description; (3) policy prescription; and (4) policy evaluation. In order to evaluate the effect of a given policy on a managed system, a "laboratory" system must be devised. The requirements of this abstracted system are that it be a selectively simplified model of the real world system whose behavior retains the main characteristics of the real system. It must be representative of the real system in its response to changes in the state of the system, whether naturally induced or dictated by a management strategy.

There is a wide range of procedures and models available which fulfill the above requirements. The techniques range from the "quick-and-dirty" to highly sophisticated, elaborate computer models. However, the recommended approach is an initial analysis based on simple procedures in order to develop an understanding of the problem; this will guide the application of more complex techniques where required.

The selection of the appropriate procedure depends upon both the available resources and the nature of the problem. Water quality problems are generally classified as to the following types:

(i) public health risks:
- body contact
- drinking water supply

(ii) eutrophication

(iii) DO depletion

and (iv) a general category consisting of conflicts of use, siltation and aesthetic considerations. The present edition of this manual is concerned mainly with techniques enabling planners to assess problems in categories (iii) and (iv). These are techniques which have proven their value in providing a unified methodology for decision making with demonstrated effectiveness.

1.2 Major Categories of Pollutants

The various substances introduced by the activities of man to aquatic environments can be classified as physical, chemical or biological in character. However, in the assessment of water quality a more convenient classification is the following:

thermal pollutants (iv)
oxygen demanding materials (iii)
nutrients (iii), (ii)
toxic substances (i) (iv)
bacteria and viruses (i)
hazardous wastes (i)

The latter category includes heavy metals, radioactive wastes, industrial organic compounds and pesticides. (The number(s) following the pollutant type refer(s) to the water quality problem designation described in section 1.1.)

Depending on the characteristics of the pollutant and of the aquatic environment, the time and space scales associated with the impact of the pollutant may vary considerably. Figures 1-1 and 1-2 show typical time and space scales associated with the impact of various

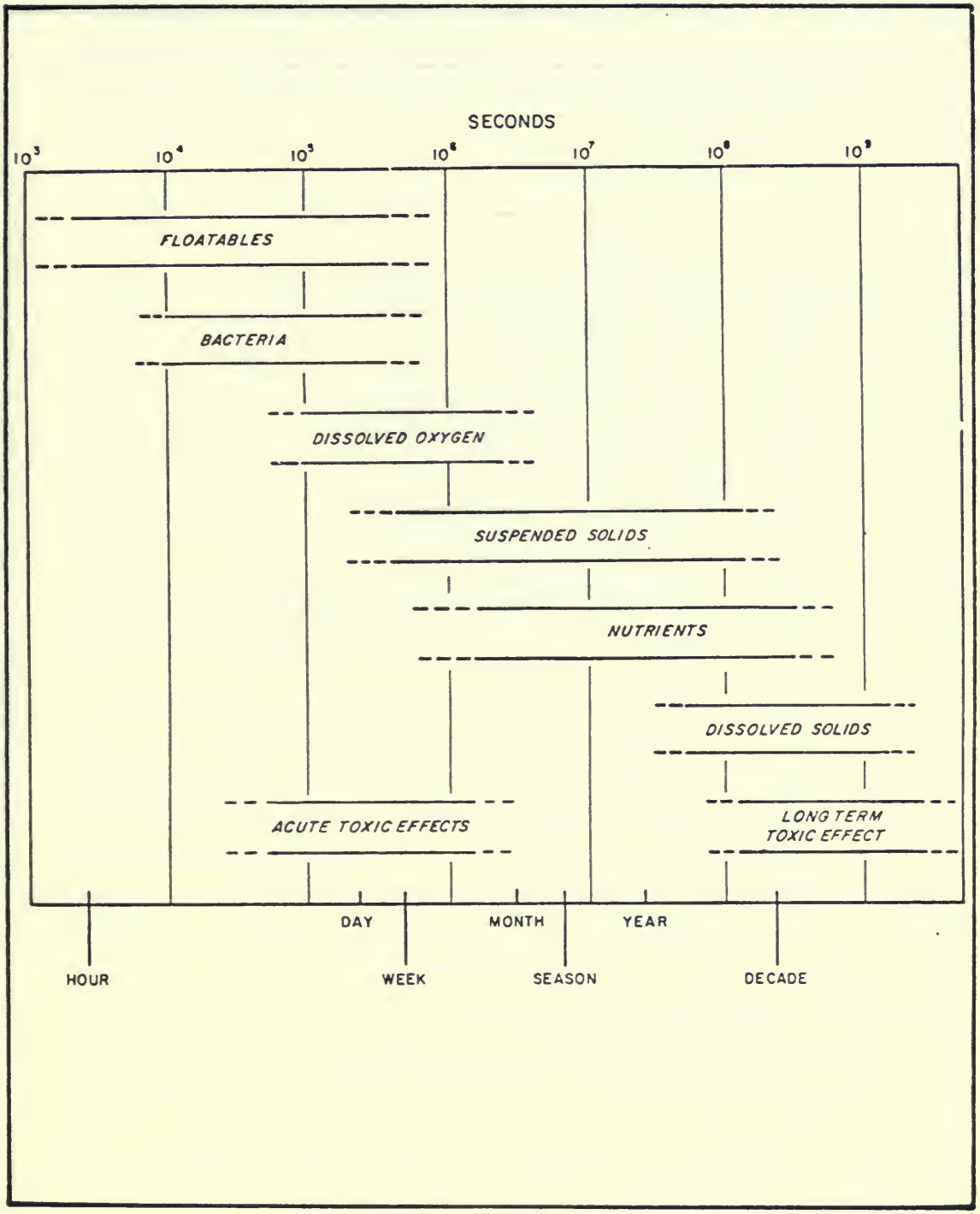

FIGURE 1-1
TIME SCALES
STORM RUNOFF WATER QUALITY PROBLEMS

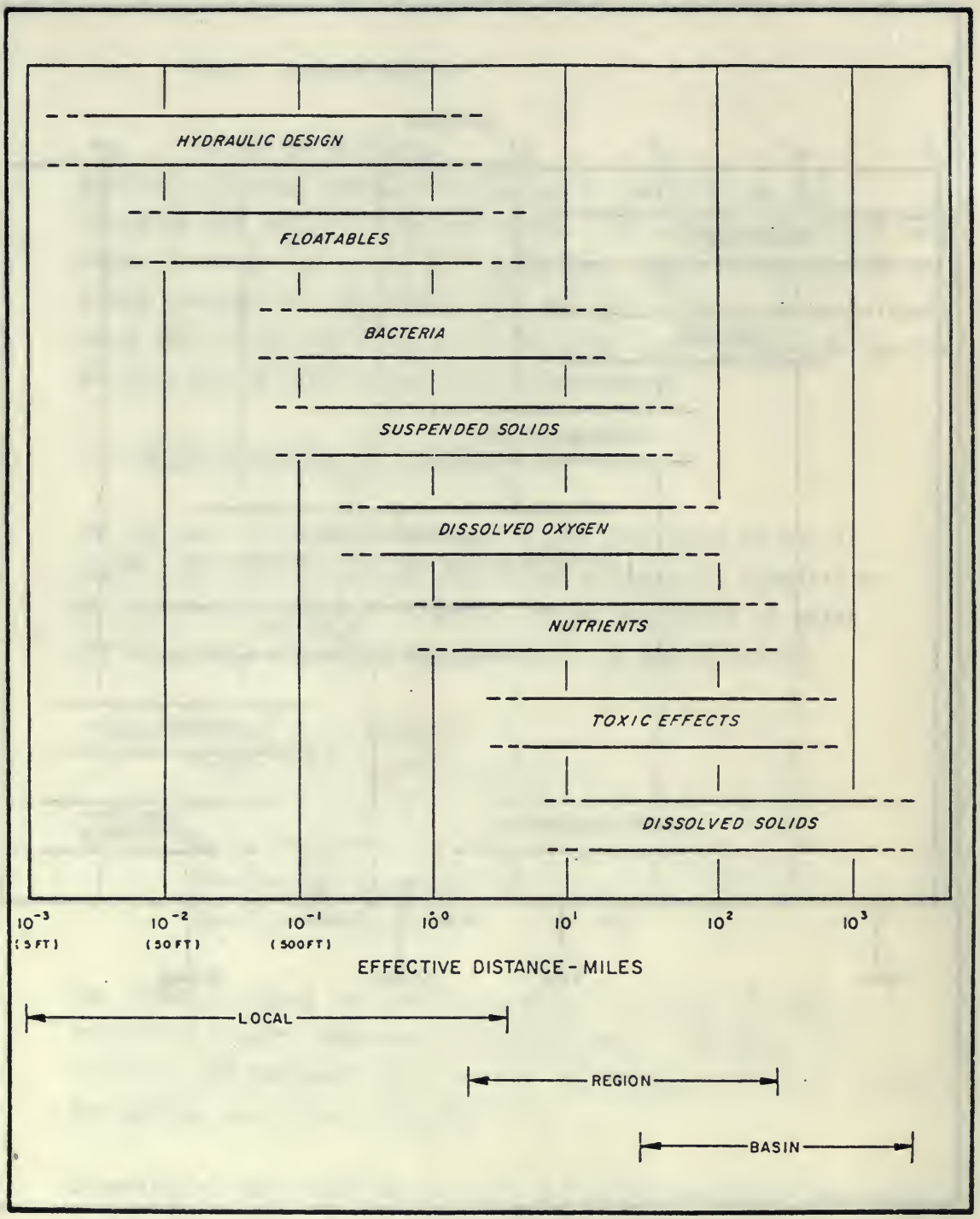

FIGURE 1-2
SPACE SCALES
STORM RUNOFF WATER QUALITY PROBLEMS

pollutants originating with stormwater runoff (1); the temporal and spatial effects of pollutants originating with various other point or diffuse sources are similar. The present edition of the manual is devoted to assessment procedures for two of the above categories of pollutant, namely, the oxygen demanding materials, and toxic substances.

1.3 Sources of Pollutants

The word "pollutant" has been used in various ways, with various connotations and implied definitions. Government is faced with the problem of providing an accurate definition for the term, in order that water quality may be maintained at an acceptably high level through clearly stated legislation and guidelines. Warren (2), basing his definition on a survey of various U.S. agencies, definitions and on case histories, defines water pollution as:

> "... any impairment of the suitability of water for any of its beneficial uses, actual or potential, by man-caused changes in the quality of the water".

It is conceivable that some waters, although unaffected by man's activities, may be unsuitable for certain beneficial uses. For example, surface water with high natural sulphur or iron content may not be satisfactory for a brewing industry, unless treated to remove the undesireable elements. Because of the social implications of man-caused changes in water quality, the distinction must be made between this type of impairment and true pollution.

Man's activities usually result in the formation of high concentrations of a pollutant. A majority of the anthropogenic pollutants in water bodies are derived from the following sources:

- Municipal and industrial effluents
- Urban runoff
- Rural runoff (including livestock and poultry wastes)
- Mining wastes

The nature of the source of a pollutant, viz. point or non-point origin and specific cause, must be considered in the impact assessment in conjunction with the various processes affecting the transport of the pollutant. This is part of the preliminary problem identification process, and is of course essential in order to ultimately prescribe and evaluate remedial policies.

1.4 Instream Influences on Water Quality

The various processes and factors affecting water quality in surface waters can be broadly classified into three categories: physical, chemical and biological phenomena. These phenomena are complex, consisting of a number of related processes and factors. The phenomena selected for consideration in an investigation are largely determined by the nature of the pollutant and its behaviour in the aquatic environment as well as the objectives of the study; the transformation or decay which the pollutant undergoes will also govern the selection of process parameters.

Some of the important processes are listed below:

Physical: convection, diffusion, evaporation, adsorption, flocculation, sedimentation, scour, gas transfer, thermal phenomena, light attenuation

Chemical: hydrolysis, ionization, chemical transformation, reaction kinetics

Biological: growth and death of viruses, bacteria, molds, fungi, algae, fish and other aquatic organisms; photosynthesis and respiration

A complete description of these processes is beyond the scope of this manual. The modeller of an aquatic system should however, be aware of the phenomena affecting the pollutant of concern. By maintaining an ecosystem outlook, there is less likelihood of making an over-simplification in the development and application of a water quality model. The processes and factors influencing the transport and decay of specific pollutants are described in subsequent sections of the manual.

1.5 Factors Affecting Dissolved Oxygen (DO)

The dissolved oxygen regime of a river is dominated by several related biochemical processes and physical influences. Brief descriptions of the most important of these factors are presented below.

Climate and Geomorphology

Instream processes are inseparable from the effects of climate and geomorphological factors. Periods of glaciation occuring between 10,000 and 100,000 years ago have moved and deposited vast quantities of materials and shaped our major river basins. The legacies of these periods of glaciation have a major influence on the water quality of the provincial streams and rivers, even today. The effects of climate and geomorphology on the instream processes are reflected by easily indentifiable aspects such as quantity and variation of flow, streambed composition, depth, velocity, turbulence, sunlight, temperature and basin vegetation.

Quantity and Variation of Flow

Quantity and variation of flow are controlled by the amount of precipitation incident on the basin and by the basin's characteristics. Streamflow is very important as it dilutes and transports the waste. The amount and seasonal distribution of flow is a major factor in determining the assimilative capacity of a stream.

Processes such as reaeration, mixing and dispersion, respiration, etc., which directly control the DO regime of a river, are a function of channel characteristics such as width, depth, velocity, channel roughness and slope.

Sunlight

Sunlight provides energy to the river system. This input varies with season of the year, hour of the day and latitude. Sunlight is employed by green plants as an energy source to fuel the process of

photosynthesis. A higher light level (to a certain maximum limit) results in higher net O_2 production, and very low light conditions can result in net O_2 depletion through plant respiration.

The ultraviolet portion of sunlight kills exposed bacteria; however, under favourable environmental conditions they are rapidly replaced.

Temperature

The temperature of a river depends primarily upon the solar energy it receives; secondly the amount of heat it can store or dissipate is dependent on its channel geometrical and hydraulic characteristics such as the surface area across which heat is exchanged and the turbulence level. Energy gain depends on available shading, angle of incidence of the sun's rays and the reflectivity and transparency of the water column. Heat loss occurs mainly through evaporation and convection. Heat can be added to the system by the infusion of hot water wasted from thermal generating plants. In general, the vertical distribution of temperature of the water is uniform in shallow streams but it may be non-uniform in deep streams with low vertical mixing. Shallow streams are more prone to rapid changes of temperature and tend to reflect ambient air temperatures.

A high proportion of groundwater flow, and shading by bank vegetation produces cooler conditions in some reaches of a stream. Higher temperature generally accelerates biological and biochemical reactions: BOD decays more rapidly and plant respiration increases; gas transfer increases at the air-water interface and oxygen solubility is reduced. The converse is true at lower temperatures. Thus, in most cases higher temperature places increased demand on the stream's oxygen resources.

Certain characteristic freshwater floral and faunal communities are associated with particular temperature ranges. In cooler water, trout and other salmonids are found in association with a diverse community with a stable oxygen regime. At higher temperatures a progressively less desirable biota is found with dominance by fewer species and with a more unstable oxygen regime.

Basin Vegetation

Vegetation, whether in-stream or on the land, can have a significant effect on in-stream processes. Land vegetation affects the rate and amount of runoff. Abundance and type of bank vegetation also controls bank stability and erosion. Trees on a bank tend to cause channels to narrow and deepen; they shade and cool the stream and reduce the washoff of surface materials.

Dense growths of aquatic plants and algae can impede streamflow, thereby reducing velocities and increasing water depth. Plants also abstract dissolved nutrients and other materials such as heavy metals from the stream. This will tend to reduce their availability further downstream. However, when plants and algae die, these materials may be released back to the water column or deposited on the stream bed. Particulate material may also be adsorbed on plant fronds.

Nutrients

Aquatic biota require a minimum supply of nutrients to sustain life and allow growth. The nutrients include nitrogen, phosphorus, oxygen, carbon, trace amounts of metals such as iron, copper and other substances. The abundance of nutrients in natural streams is related to the geology and physiography of the watershed. In the Canadian shield areas carbon availability is limited and therefore the growth of aquatic plants and algae is controlled by this element, although other nutrients are abundant. On the other hand, phosphorus appears to be the limiting nutrient in southern Ontario streams. The abundance of nitrogen in the atmosphere and the existence of nitrogen fixing bacteria make this nutrient nonlimiting in most cases. Other nutrients are generally available in the stream environment.

Phosphorus levels above certain limits are known to stimulate aquatic plant and algal growth. However, very high levels of phosphorus (greater than 5 mg/l) can prove toxic to plant life. Plants and algae have the ability to store excess phosphorus (nutrients) to be used when the supply is too low. This is known as luxurious uptake.

pH

The pH of a system denotes the concentration of hydrogen ions (H^+) present. The numerical value of pH represents the negative logarithm of this concentration. Hydroxyl ions (OH^-) exist in balance with the hydrogen ion: as the concentration of one ion increases the concentration of the other decreases. Bacteria undergo optimum growth within a certain pH range. For example, nitrifying bacteria prefer an alkaline environment with an optimum pH range of 8 to 9. High pH values also allow NH_4^+ to be lost to the atmosphere. This may result in less de-oxygenation resulting from nitrification reactions; offsetting this, however, high pH also increases the toxicity of free ammonia (NH_3).

Many natural processes produce end products which alter the pH. This results in an enhancement or retardation of the process. However, a stream's natural buffering capacity (i.e. ability to accept hydrogen or hydroxyl ions) generally reduces the influence of these end products on the pH value. Nuisance levels of aquatic plants have a strong diurnal effect on stream pH even in well buffered systems by altering the levels of DO, and CO_2. The absorption of CO_2 into solution drives this process through the production of carbonic acid.

Toxicity

Toxic effects on the biota such as those brought about by the presence of chlorine, heavy metals, pesticides, radionuclides, H_2S, free ammonia, industrial organics and elevated or depressed pH levels generally tend to delay the oxidation of materials. In cases where food and nutrients are plentiful the biota may develop a tolerance to the toxic element or a different more tolerant organism may take over. Toxic materials tend to delay biochemical oxygen demands of organic materials. This effect causes the zone of oxygen depletion to be located further downstream.

General Water Quality Effects

A healthy biota is found where good water quality with a moderate amount of food and nutrients prevails. Aquatic plants may give way to sewage molds under conditions of high organic load. Generally the less complex the biota the easier it can tolerate excessive organic enrichment. In fact, attached algae prefer clear, cool flowing water with low levels of oxidizable organics and moderate amounts of nutrients while bacteria flourish in rich organic sludges with high nutrient levels. High levels of free ammonia however, are toxic to nitrifiers. Bacterial population dynamics in the presence of a high BOD load can be very unstable and therefore can be markedly different after treatment is instituted.

Man's Influences

Man generally attempts to control the volume and distribution of runoff (flooding and water supply) and channel alignment (for erosion prevention) in a river basin. Flood volume control generally leads to construction of dams. Dams tend to reduce mixing in the stream, encourage sediment deposits, raise stream temperature, increase evaporation and force change in the natural biota which can lead to an alteration of assimilative capacity. Channel straightening tends to increase velocities and lower stream depths; this increases the assimilation capacity by improving mixing throughout the reach affected, and by lowering the residence times of pollutants. Flow diversions from or to a river basin can considerably effect the volume of flow available to carry the waste. Denudation of a riverbank tends to increase stream temperature by allowing greater access of sunlight, and to increase bank susceptibility to erosion.

All the factors previously described have a major effect on the assimilative capacity of a stream. Some of these effects can constrain similar streamflows to widely differing assimilation capacities or vice versa. For example, the drought flow assimilation capacities of the Kapuskasing and Abitibi rivers are very close but the drought flow of the Abitibi is ten times higher

than that of the Kapuskasing. The assessment techniques presented in this edition of the manual deal with determining the sum of these effects by means of various more easily identifiable and quantifiable parameters such as depth, velocity, sunlight availability, temperature and biochemical decay rates.

1.6 Processes Affecting Instream Dissolved Oxygen

The DO regime of a river is affected by a large number of natural and man-made forces. Although many of the forces acting on the system cannot be directly controlled, it is necessary to understand their effects on stream environment in order to make a rational assessment of the assimilative capacity of the stream.

Dissolved oxygen is important in maintaining a healthy stream environment that will provide for the widest variety of possible uses. Each species of fish has specific oxygen requirements, and these requirements change through the life cycle. For example, an average DO level of 6 mg/l is required for rainbow trout and spawning grounds of warm water fishery. Conversely, very high DO levels (15-20 mg/l) can be harmful to fish life if exposure is prolonged.

If the stream becomes anoxic, all aerobic life will die. Heavy metals and nutrients will be released from the sediments and noxious odours will result. It is therefore important to maintain adequate dissolved oxygen levels in our watercourses so that full aesthetic enjoyment, recreational and water supply use can be maintained.

Dissolved oxygen occurs naturally in stream waters and is derived from the atmosphere. The air-water contact allows the gases found in the atmosphere to be dissolved in water according to their partial pressure in the atmosphere and relative solubility in water. The water's ability to retain dissolved gases is directly proportional to pressure and inversely proportional to temperature. For example, at $0^{o}C$, the saturated DO concentration is 14.6 mg/l, while at $25^{o}C$, it is 8.4 mg/l in fresh water at standard

atmospheric pressure.[1] The difference between the saturation value and the ambient DO concentration is termed the dissolved oxygen deficit. The ambient DO concentration may exceed saturation, in which case, the oxygen deficit has a negative value.

The various processes directly affecting the dissolved oxygen deficit can be classified into two major categories: sources and sinks.

A. Sources of Dissolved Oxygen

A source is defined as any process that adds oxygen to the stream, such as photosynthesis.

Natural Reaeration

The stream's primary source of dissolved oxygen is natural reaeration, which is a physical phenomenon. Reaeration occurs as a result of molecular diffusion of oxygen through the thin surface film followed by turbulent mixing through the water column. The turbulence in the stream is related to water velocity, stream depth, and roughness of the channel, as well as wave action caused by wind.

The transfer of oxygen can be either positive, (i.e. to the water) or negative (i.e. to the atmosphere), depending on whether the gas concentration in water is at a higher or lower concentration than that found in the atmosphere (3). The stream's reaeration rate is also dependent on the temperature, concentrations of dissolved materials, and degree of inhibition at the air-water interface caused by the presence of surface scums or films.

Photosynthesis

The photosynthetic process of green plants and algae is a significant source of oxygen to waterbodies. This source is only active during daylight hours and varies with available sunlight

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1 standard atmosphere = 101.35 kPa (kilopascals)

intensity, water clarity, nutrient availability and season of the year. If oxygen levels are elevated above the concentrations that can be dissolved in water, then some proportion of the oxygen that exceeds this level will be lost to the atmosphere through the physical process of gas transfer referred to above.

Inflows to the System

Inflowing tributary streams influence the oxygen balance of a river system either positively, with oxygen-rich water, or negatively, with water deficient in oxygen or carrying a high proportion of oxygen demanding substances. The relative importance of this oxygen input is proportional to the ratio of volume and quality of the tributary input to that of the receiving system.

Artificial Aeration

Oxygen may be induced artificially by increasing the surface area of water in contact with the air by such means as allowing free fall of water over a dam or over a series of shallow weirs. The latter also spreads the reaeration process over a longer reach. Compressed air or surface agitators could also be used to introduce oxygen into the system. In practice, free falls over dams are very common; however, dams are not usually specifically designed for this purpose.

Other Sources

Processes such as denitrification, where nitrates are processed by anaerobic bacteria to nitrites and to free nitrogen, can result in the release of oxygen into the stream. Similarly, the reduction of sulphates may add oxygen to the stream. The contribution of oxygen from such sources is usually negligible. These reactions are relatively rare, occurring in certain environments such as the top layer of sediments or in the water column in deep channels.

B. Sinks of Dissolved Oxygen

An oxygen sink is defined as any process that removes oxygen from the stream, (e.g. bacterial respiration). The major oxygen sink in

polluted waters results from biochemical decay of carbonaceous and nitrogenous organic substances. Algal respiration is also a major, although transitory oxygen sink in some waters.

The process of biochemical decay is complex and its effect depends to a large degree on the nature of the organic substances, and other transformations occurring, particularly changes in nitrogen compounds.

The complexity and importance of these processes warrants a more detailed explanation than that given below for other oxygen sinks, and therefore these are discussed separately in sections 1.7 and 1.8.

Oxidation of Inorganic Chemicals

Certain inorganic substances such as sulphides (S^{-2}), sulphites (S^{+4}), ferrous compounds (Fe^{+2}), etc., can utilize dissolved oxygen through chemical reactions. These reactions can cause an instantaneous oxygen demand and usually occur immediately below waste outfalls. In these cases in-situ measurements of DO are required. For this procedure, refer to "Standard Methods for the Examination of Water and Wastewater"(4).

Immediate oxygen demand may occur in the sample bottle, and the reaction complete before the sample reaches the laboratory. In this event, field testing is important.

The chemical oxygen demand (COD) test in the laboratory measures all the oxygen required to stabilize a waste, including CBOD, NBOD and the oxygen demand of inorganic chemicals. However, in a stream under natural condition, a portion of the oxygen demanding materials present may not undergo oxidation.

Benthic Oxidation

Benthic oxidation which is represented by the sediment oxygen demand (SOD), arises from the settling and adsorption of suspended oxidizable materials onto the benthos or plants. This condition occurs when stream velocities are insufficient to maintain these

materials in suspension as well as in the presence of surfaces to which they could become attached. The surface benthic biota (sewage molds, bacteria, etc.) process the food material aerobically by utilizing oxygen from the stream. Further decomposition takes place anaerobically in the benthic subsurface layer. The anaerobic decomposition produces waste gases which might rise through the water column and remove oxygen through purging action. If the rise of gases is violent enough, settled materials may be resuspended in the water column and thereby increase the BOD load. The disturbance of the sediment surface also breaks up partially stabilized organic material and may therefore greatly enhance sediment oxygen demand.

Plant Respiration

As previously mentioned, plant photosynthesis occurs only during periods of sunlight. Under this condition, a net increase in stream dissolved oxygen results. Plants continually respire however, and thus, on days of low sunlight, and at night, plants create another oxygen sink in terms of net effect on the stream DO regime. During days with enough sunlight for an overall net production of oxygen, plants still act as a DO sink during the night-time period. This nocturnal sink can have a damaging impact on aquatic communities in cases where near complete DO depletion prevails for 4 to 6 hours.

1.7 The DO - BOD Relationship

By far the most serious impact of organic wastes on the aquatic environment is the great quantity of oxygen required to degrade the material, often far in excess of the natural capacity of the water to dissolve free oxygen to replace that used. The BOD (Biochemical Oxygen Demand) test is a measure of the strength of a waste water in terms of its ability to deplete the naturally available dissolved oxygen in this way. BOD is defined as the quantity of oxygen required, for a period specified, for stabilization of decomposable organic matter and oxidizable inorganic matter by aerobic biological activity. A complete description of the biochemistry of the process of aerobic degradation of waterborne organic waste is beyond the scope of this manual. However, the following overview should provide sufficient information on the most relevant aspects related to waste assimilation studies.

The two major organic materials commonly encountered in streams are basically carbonaceous and nitrogenous in composition. These materials are fairly rapidly metabolized by bacteria under aerobic conditions. Carbonaceous materials are degraded by heterotrophic bacteria, i.e. bacteria which derive energy from that stored in organic chemical bonds, with oxygen and carbonaceous compounds as inputs to the process, and with CO_2 and water as end products. After a supply of organic substrate is exhausted in this process, more oxygen is consumed by these bacteria in the process of endogenous respiration. During this process, a bacterium consumes stored energy-rich chemical compounds and finally its own structural cellular material. The oxygen consumed during this entire process is known as the carbonaceous biochemical oxygen demand, or CBOD.

Nitrogenous materials are metabolized in a variety of ways by certain groups of specialized bacteria, with the process of nitrification (discussed in section 1.8) the most important of these in terms of oxygen uptake. The oxygen demand resulting from nitrogen oxidation is termed NBOD (nitrogenous biochemical oxygen demand, or simply NOD).

Together NBOD and CBOD comprise the total oxygen demand (TOD). When reference is made to BOD it must be assumed that it is the total oxygen demand to which reference is made, unless specified otherwise.

The evaluation of BOD is carried out in the laboratory by incubating a water sample at $20^{o}C$ and measuring its oxygen uptake at intervals. Normally, the oxygen uptake is monitored for five days and the result reported as BOD_5 or five-day BOD (at 20^{o}). The five-day BOD is the traditional measure of the strength of a waste, because the time of travel of most rivers in England, where the BOD concept originated, is about five days.

Several factors complicate the interpretation of BOD_5 test results. The inference of a certain oxygen demand exerted in the stream over five days may be misleading. Firstly, the rate at which the material is oxidized in the stream may be quite different from the rate at which it is oxidized in the laboratory (see Figure 1-3). Most stream rates are higher than those determined in the lab. Some of the factors which affect the instream rate are:

(1) Turbulence: The effect of turbulence is to increase contact between bacterial growth sites and hence increase the speed of biochemical reactions;

(2) Biological growths: growths attached to the stream bed provide ideal conditions for many micro-organisms, increasing the rate of BOD decay;

(3) Immediate demand: some wastes contain strong reductants which exert an immediate oxygen demand in the stream;

(4) Nutrients: some wastes, or receiving waters are deficient in nitrogen or phosphorous, causing a slower decay rate than observed in the laboratory with standard dilution water.

Because of the difference between laboratory and in-stream decay rates, in-stream rates must be directly measured. Samples are usually taken at distance intervals below the pollutant load source in a stream and analyzed for BOD_5. Every point on a plot of stream BOD_5 versus distance will have a corresponding point on a laboratory curve of BOD versus time.[1] (see Figure 1-3)

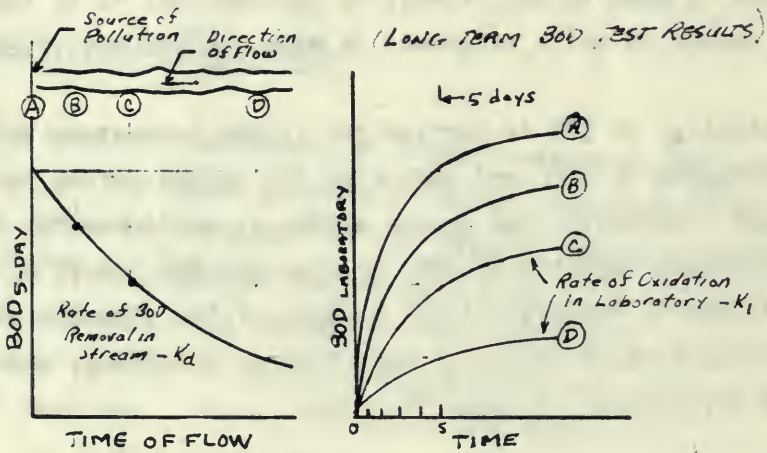

Figure 1-3
Relationship Between Stream and Laboratory BOD Decay Rates

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1 Analysis of these results gives the in-stream first order decay coefficient K_d (see section 4.5.1).

A second factor complicating the interpretation of BOD_5 involves the intrinsic rate at which organic materials are potentially degraded, irrespective of the rate at which the environment can oxidize the material. Certain slowly degradable materials may exert their effects on a river system's DO regime over a much longer period than five days. Pulpwood waste is a well known example in Ontario. For this reason a waste water is best characterized by a long term BOD test where possible. The total oxygen demand of the carbonaceous organics in the waste water is measured by a long term (20-30 days) BOD test, and is known as the ultimate BOD (BOD_u). The determination of an ultimate BOD, based on this long-term BOD analysis, is discussed in section 4.4.2.

A third consideration in interpreting BOD_5 test results involves suspended organic substances. Oxygen demanding materials may be physically removed from the water column by settling, or added to the water column by scouring. Station by station analysis of water samples for volatile suspended solids will help identify this form of BOD removal. Again, this factor is often prevalent in Ontario streams where the effluent is of the nature of pulpwood waste.[1]

Some final comments are appropriate on the measurement and interpretation of NBOD and CBOD. Nitrification may or may not be a significant process in certain stream situations. Modern sewage treatment plant facilities are now being designed for an in-plant nitrification of secondary effluent. Under this level of treatment, and in situations where effluent is low in organic nitrogen, NBOD will be correspondingly low. However, downstream of more conventional treatment facilities, lagoons, etc., NBOD may comprise up to 70% of the total oxygen demand. The determination of NBOD in any of these situations is best accomplished by analysis of station by station water samples for the important nitrogen forms.

Nitrification in laboratory BOD tests may not always occur until after a significant lag period has elapsed; this is related to a period of growth of nitrifying bacteria which may be in much lower

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1 In steady state DO - BOD modelling, removal of BOD through settling is characterized by the process rate K_r, the first order removal rate coefficient (see section 4.5.1).

densities than would be found in the stream environment. The present trend is for laboratory BOD analysis using chemical inhibitors of nitrifying bacteria, so that only CBOD is determined. By whatever means accomplished, differences in the characteristic rates at which nitrogenous and carbonaceous substances decay in a stream dictate that these two components of BOD be separated in any waste assessment study.

1.8 Nitrogen Transformations Affecting DO

Nitrogen parameters are essential to include in water quality assessment for several reasons. Un-ionized ammonia (NH_3) is a highly toxic substance; nitrate and nitrite may be present in sufficiently high concentrations to represent a danger to man and to livestock drinking this water. Infants and pregnant women, are particularly at risk, as high nitrate in the drinking water supply has been shown (5) to cause methemoglobinemia[1]. For this reason the Ministry has set a criterion of 10 mg/L nitrate for raw water supply.

In terms of dissolved oxygen effects, ammonia is of importance in that under normal stream conditions, it is readily oxidized to nitrate by the process of nitrification. Several biochemical reactions control nitrogen cycling in the stream environment (figure 1-4). Sinking of organic-N will be found where settling of untreated nitrogenous wastes occurs after discharge to the water body. Dentrification is a bacteriological process which may occur in stream sediments, particularly those high in carbonaceous matter. Oxidation of this material by facultative anaerobic bacteria produces reducing conditions under which denitrifying bacteria reduce nitrate to nitrogen gas:

$$4H^+ + 4NO_3^- \xrightarrow{\text{bacteria}} 2\ N_2 + 5\ O_2 + 2\ H_2O$$

Other stream processes which may contribute to the removal of mobile N from the system are plant uptake and immobilization in sediments.

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1 A disease causing defective heart structure, resulting in the "blue baby" condition.

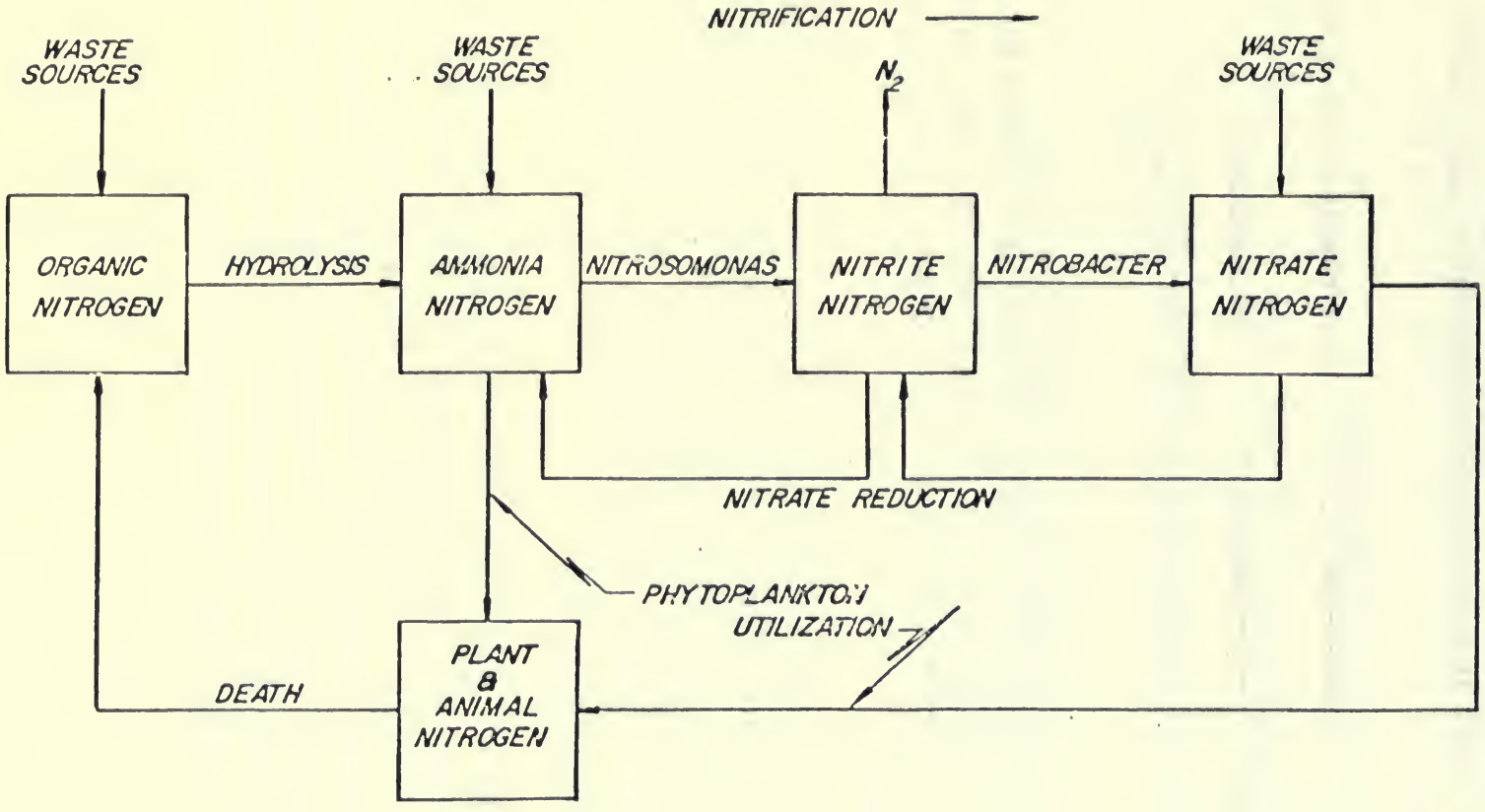

Figure 1-4
Nitrogen Transformations Affecting DO

Of the reactions depicted in figure 1-4, the most important effects on the stream DO regime usually result from the nitrification process. This is a two step process involving several species of bacteria:

(1) $2NH_3 + 3O_2 \longrightarrow 2HNO_2$ (Nitrococcus; Nitrosomonas)

(2) $2HNO_2 + O_2 \longrightarrow 2HNO_3$ (Nitrobacter)

In the first step, free ammonia is oxidized to nitrite by bacteria such as Nitrosomonas, and in the second step, the highly unstable nitrite is further oxidized to nitrate by species such as Nitrobacter.

During the process, a large amount of dissolved oxygen is consumed: 3.43 mg per mg of ammonia nitrogen in the first step and 1.14 mg per mg of nitrate in the second step[1]. The rate at which the process occurs has been shown to be independent of DO concentrations above 2 mg/l, and is usually treated as first order with respect to substrate concentration.

The uptake of oxygen by nitrogenous compounds is referred to as NBOD (Nitrogenous Biochemical Oxygen Demand) or simply NOD. Water sample analysis is usually done for ammonia, nitrate + nitrite, and total Kjeldahl nitrogen (TKN); the latter parameter includes organic-N and (NH_3 + NH_4). In modelling procedures, the uptake of DO is estimated stoichiometrically[1] by considering 4.57 moles (3.43 + 1.14) of oxygen are consumed by one mole of TKN.

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1 By considering chemical reactions only.

1. Anon; 1976; Areawide Assessment Procedures Manual, Vol. 1; Municipal Environmental Research Laboratory Office of Res. and Dev.; U.S. E.P.A.; Cincinnati, Ohio; Chapter 3.

2. Warren, Charles E.; 1971; Biology and Water Pollution Control; W. B. Saunders Co.; Chapter 2, pp. 15-23.

3. Gordon, M. F., J. Geyer, D. Okun; 1968; Water and Wastewater Engineering, Vol. II; John Wiley and Sons, Inc.; Chapter 23.

4. Anon; 1976; Standard Methods for the Examination of Water and Wastewater; 14th Edition; APHA, AWWA, WPCF; Washington, D.C., U.S.A.

5. Walton, G; 1951; "Survey of the Literature Relating to Infant Methemoglobinemia due to Nitrate Contaminated Water", American Society of Health; Pub. Health Eng., Abstr. 31.

CHAPTER II

WATER QUALITY MODELS

CHAPTER TWO

DISSOLVED OXYGEN MODELS

2.1 Introductory Note

All users of the models subsequently described in this manual should be familiar with their limitations and the assumptions upon which they are based. The assumptions are summarized in the present chapter. In addition, the theory underlying the basic water quality models is developed. This is included for the benefit of the user who wants a more complete description of the development and formulation of some of the models used in water quality assessment, including the MOE-Steady State DO-BOD model.

2.2 Model Development

This section describes the basic transport equation which provides the foundation of various models including DO-BOD models, mixing zone models, and models of transport of various pollutants.

2.2.1 Development of Conceptual Models

The various processes affecting water quality in streams, lakes and estuaries can be broadly classified as physical, chemical and biological. Some of the main processes are:

1. Physical- convection, diffusion, evaporation, sedimentation, gas transfer and thermal phenomena.

2. Chemical- solution equilibria, reaction kinetics.

3. Biological- microbial metabolism, plant photosynthesis and respiration.

As in all living systems, the processes of the aquatic system are complex, involving interrelated phenomena such as bio-physical factors, and virtually thousands of chemical and biochemical transformations. Mathematical models provide a means of dealing with such complexity to achieve specified management goals.

A mathematical model is a linked set of arithmetic equations which represent the various processes within the prototype system. It is an abstraction of reality and in this sense is a simplication of the real world which forms the basis of the model. The degree of simplification depends on both the intent of the modeller and the depth of his knowledge of the real world. The various processes that are not well understood may be treated as "black boxes". In this approach, the overall effect of the process is represented by equations which are empirically derived or based on fundamental relationships.

The basic transport equation is a theoretical description of change in any three dimensional system and may be applied to the aquatic system. The equation incorporates the effects of all physical, chemical and biological processes, and is developed by constructing a materials balance about an elemental volume, employing the principle of conservation of mass:

$$\underbrace{\frac{\partial c}{\partial t}}_{\text{Local time Variation}} + \underbrace{u_x\frac{\partial c}{\partial x} + u_y\frac{\partial c}{\partial x} + u_z\frac{\partial c}{\partial z}}_{\text{Convection Terms}} = \underbrace{E_x\frac{\partial^2 c}{\partial x^2} + E_y\frac{\partial^2 c}{\partial y^2} + E_z\frac{\partial^2 c}{\partial z^2}}_{\text{Dispersion Terms}} \pm \underbrace{\Sigma S}_{\text{Sources \& Sinks}} \qquad (1)$$

where:

c	=	local concentration at (x, y, z) at time t;
E_x, E_y, E_z	=	dispersion coefficients in x, y, z directions, respectively;
u_x, u_y, u_z	=	mean velocities in x, y, z directions respectively;
ΣS	=	sources and sinks term.

The physical meaning of terms in Equation 1, and their significance in river water quality model development are discussed below.

Local Time Variation:

This term is associated with the concentration fluctuations at a point with time. It is of importance when variations occur in the waste loading (e.g. fluctuations in BOD input loading), river flow rates (e.g. resulting from stormwater runoff), and/or process parameters (e.g. photosynthetic rate).

Convection Terms:

In natural rivers the flow is mainly unidirectional; hence, convection in the x direction only will be significant. However, in streams of irregular geometry, convective terms in the y and z directions may be significant.

Dispersion Terms:

The dispersion terms become important whenever there exist significant concentration gradients in any direction. A practical example is the effluent dispersion from bank outfalls in shallow streams, requiring the inclusion of the transverse dispersion term. In lakes, usually dispersion in two of three directions needs to be included.

Source-Sink Term:

The sources part of this term includes point and non-point pollutant inputs to the water body. The sinks part includes the elimination of pollutants through various processes such as biochemical decay.

2.2.2 Development of DO-BOD Models

As stated above, the basic transport equation provides the basis for various types of models. In this section, equation 1 is taken as

the starting point in the development of the steady-state DO-BOD model used in the Ministry, and subsequently described in more detail.

The basic equation 1 has the following the limitations:

a) There is no general, analytical closed-form solution.

b) Only approximate numerical solutions (finite difference and finite elements) are available, limiting its practical usage.

c) Large amounts of data would be required for its calibration and verification.

d) It would be costly to perform calculations even on a computer due to large storage requirements and compiler time.

The model may be reasonably simplified by assuming that only convection in the x direction is significant and that diffusion effects are negligible; equation 1 thus reduces to:

$$\frac{\partial c}{\partial t} + u_x \frac{\partial c}{\partial x} = \pm \Sigma S \qquad (2)$$

If it is further assumed that a condition of "steady-state" prevails, that is, there is no change in streamflow, temperature, waste loads and stream processes, then local time variation will be nil; i.e., $\frac{\partial c}{\partial t} = 0$. As a result, equation 2 reduces to:

$$u_x \frac{\partial c}{\partial x} = \pm \Sigma S \qquad (3)$$

Streeter and Phelps Model

Since $\frac{u_x}{\partial x} = \frac{1}{t}$; where t=time of travel, equation 3 may be modified to:

$$\frac{dc}{dt} = \pm \Sigma S$$

The "c" term of this equation may represent the concentration of DO, BOD, or any other water quality parameter.

If the parameter of interest is biochemical oxygen demand, then ΣS would be comprised of sources of oxygen demanding pollutants, and sinks including decay by aerobic bacteria.

If we substitute L to represent ultimate BOD and consider only decay, the above equation may be re-written:

$$\frac{dL}{dt} = -K_d L \qquad (4)$$

where:

L = ultimate BOD concentration, mg/L;

K_d = instream first order decay rate, per day (to base e).

Equation 4 is based on the assumption that the decay of BOD in the stream follows a first order kinetic law. The term dL/dt represents the change in BOD concentration with respect to the time of travel. The BOD removal in a stream takes place as it travels downstream at a speed equal to the mean velocity of water (u). The time (t) required to travel a given distance (x) in the river is thus equal to x/u. Therefore, the left hand side of equation 4 can also be written as:

$$u \frac{dL}{dx}$$

It should be remembered that the time t in equation 4 refers to the time of travel and hence should not be confused with the time of day.

Equation 3 can also be written in terms of oxygen deficit. The word "deficit" describes a condition in the stream when the observed dissolved oxygen is found to be different from that which would be expected at 100% saturation under the prevailing conditions of atmospheric pressure, temperature and concentration of dissolved materials found in the system. Equation 3 will then take the form:

$$\frac{dD}{dt} = K_d L \qquad (5)$$

where:

$D = c_s - c$ = the dissolved oxygen deficit in the reach, mg/L;

c_s = saturation DO value, mg/L;

c = concentration of DO in the reach, mg/L.

t = time of travel in the reach

Equation 5 is more convenient to use when describing the oxygen balance in a stream.

Generally, it is BOD_5, not L, which is reported by the laboratory. This represents a portion of L. The relation between the two parameters is:

$$L = BOD_5/(1 - 10^{-k_1 t}) \qquad (6)$$

where t equals five days for BOD_5, and k_1 represents the decay rate (base 10) obtained from a long-term BOD test of a water sample. Since it is not practicable to carry out the ultimate BOD test on all collected samples (particularly on stream water samples), the value of k_1 is obtained from one or two representative tests.

Integration of Equation 4 with the boundary condition of $L = L_o$ at $t = 0$ results in:

$$L = L_o e^{-K_d t} \qquad (7)$$

where:

L_o = Ultimate BOD value at the head of the reach where t equals zero, mg/L;

t = time of travel within the reach in days;

K_d = instream first order decay rate for the reach, per day (to base e).

In the steady state analysis, it is assumed that the waste water is completely mixed with the stream water instantaneously.[1] Therefore, L_o represents the uniformly mixed instream concentration of ultimate BOD immediately below the source and it is calculated from a mass balance of the BOD loads as follows:

$$L_o = \frac{L_e Q_e + L_u Q_u}{Q_e + Q_u} \qquad (8)$$

where:

Q_e = effluent flow rate, m^3/sec;
Q_u = flow in the river just upstream of the source, m^3/sec;
L_e = concentration of ultimate BOD in the effluent, mg/L;
L_u = concentration of ultimate BOD in the river just upstream of the source, mg/L.

At the same time as the organic matter is undergoing aerobic decomposition in the stream, oxygen is being replenished from the atmosphere. This reaeration rate is proportional to the oxygen deficit created by the decomposition of BOD. The reaeration process is represented by the following equation:

$$\frac{dc}{dt} = K_a D \qquad (9)$$

where:

c = concentration of dissolved oxygen at the end of the reach, mg/L;
D = $(c_s - c)$ = the dissolved oxygen deficit, mg/L;
c_s = saturation DO value, mg/L;

---

1 In some river situations, significant transverse gradients of BOD and DO may be present at some stations below an outfall. This region of the stream is known as the mixing zone.

K_a = reaeration rate coefficient, per day (to base e);

dc/dt = change in oxygen concentration with respect to time of travel t.

By replacing the DO concentration (c) in Equation 9 with the deficit ($D = c_s - c$), the following equation is obtained:

$$\frac{dD}{dt} = -K_a D \tag{10}$$

Integration of equation 10, with the boundary condition of $D = D_o$ at $t = 0$, yields:

$$D = (D_o e^{-K_a t}) \tag{11}$$

where:

$D_o = c_s - c_o$ = initial deficit of the head of the reach where t = 0, mg/L;

c_o = initial DO at the head of the reach, mg/L;

t = time of travel, in days.

According to Streeter and Phelps, the oxygen balance within the reach is obtained by combining Equations 5 and 10. Thus, the Streeter-Phelps model is:

$$\frac{dD}{dt} = K_d L - K_a D \tag{12}$$

By integrating Equation 12 subject to the boundary conditions of $L = L_o$ and $D = D_o$ at $t = 0$, the following oxygen sag equation is obtained:

$$D = \frac{K_d L_o}{K_a - K_d} (e^{-K_d t} - e^{-K_a t}) + D_o e^{-K_a t} \tag{13}$$

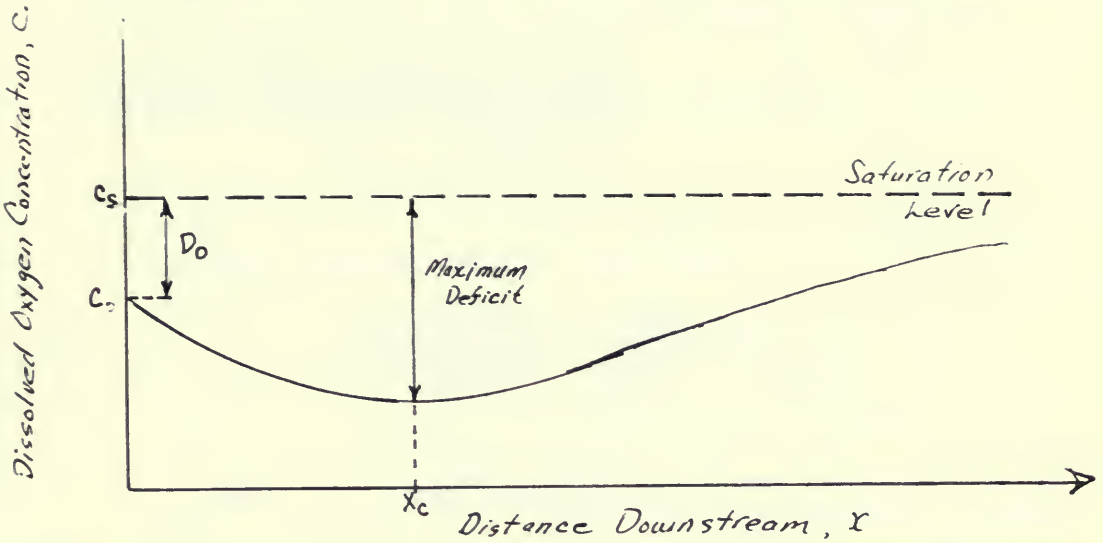

Figure 2-1
Dissolved Oxygen Sag Curve

The graphical form of the Streeter-Phelps equation is illustrated in Figure 2-1, which shows the dissolved oxygen sag curve for a river. Decomposition of the organic matter begins immediately below the point of waste discharge. This process consumes oxygen at a rate that exceeds the rate of replenishment through reaeration. The result is a gradual decrease of instream DO concentration or an increase in the deficit. Finally a point is reached at which the rate of oxygen utilization equals the rate of replenishment by reaeration (point X_c in Figure 2-1). The point X_c is generally called the critical point, and is of significance in the determination of the waste assimilation capacity of the river.

The critical deficit (D_c) at the critical point (X_c) is given by:

$$D_c = \frac{K_d \cdot L_o}{K_a} e^{-K_d t_c} \qquad (14)$$

where:

$t_c = \frac{X_c}{u}$ = the time of travel from the point of waste discharge to the critical point(X_c), in days;

u = the mean velocity of flow, m/sec.

The distance of X_c is determined from the following equation:

$$X_c = \frac{u}{K_a - K_d} \ln \left\{ \frac{K_a}{K_d} \left[1 - \frac{D_o (K_a - K_d)}{K_d L_o} \right] \right\} \qquad (15)$$

where (ln) stands for the natural logarithm (i.e., $\log_e$...).

The Streeter-Phelps model was the first dissolved oxygen model to be introduced. This model is particularly applicable to rivers where the predominant processes are reaeration and decomposition of organic matter. Modification of this classic model have been proposed to incorporate the effects of other significant processes such as photosynthesis, respiration and benthic demand.

The MOE Steady State[1] DO Model

The MOE steady state DO model considers the following processes:

a) decomposition of CBOD;
b) decomposition of NBOD;
c) benthic or sediment oxygen demand, SOD;
d) algal and plant respiration, R;
e) photosynthetic oxygen production by plants and algae, P; and
f) atmospheric reaeration, K_a.

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1 The MOE "Steady-state" model is really quasi-dynamic in that there is temporal (within-day) variation in D resulting from variation in the photosynthesis term. Photosynthesis, while assumed constant from day to day, is varied within the day to reflect the effects of photoperiod on plant uptake and production of DO. All other parameters vary only with downstream distance x, or time of travel.

The combined effect of all the above processes is given in terms of DO deficit, by the following partial differential equation:

$$\frac{\partial D(x,t)}{\partial t} + u \frac{\partial D(x,t)}{\partial x} = K_d L(x) + K_n N(x) + S(x) + R(x) - P(x,t) - K_a D(x,t) \quad (16)$$

where:

x	=	distance below source, m;
t	=	time of day, days;
$L_{(x)}$	=	$L_o e^{-K_r(x/u)}$; concentration of CBOD at distance x, mg/L;
$N_{(x)}$	=	$N_o e^{-K_n(x/u)}$; concentration of NBOD at distance x, mg/L;
L_o	=	concentration of CBOD at head of the reach, mg/L;
N_o	=	concentration of NBOD at head of reach, mg/L;
$S_{(x)}$	=	average rate of sediment oxygen demand in the reach, mg/L.day;
$R_{(x)}$	=	average rate of algal and plant respiration in the reach, mg/L.day;
$P_{(x,t)}$	=	photosynthetic oxygen production in the reach at distance x and time of day, t, mg/L.day;
$D_{(x,t)}$	=	DO deficit at distance x and time of day, t, mg/L;
K_d	=	first-order decay rate for CBOD in the reach, day^{-1} (to base e);
K_r	=	first-order removal rate through sedimentation and decay of CBOD in the reach, day^{-1} (to base e);
K_n	=	first-order decay rate of NBOD in the reach, day^{-1} (to base e);
K_a	=	first-order atmospheric reaeration rate in the reach, day^{-1} (to base e);
$\frac{\partial D(x,t)}{\partial t}$	=	rate of change in DO deficit with respect to time of day, mg/L.day
$\frac{\partial D(x,t)}{\partial x}$	=	rate of change in DO deficit with respect to distance x, mg/L.m
u	=	mean velocity of flow (in the x direction) in the reach, m/day.

Analytical Solution

If the photosynthetic rate is assumed to vary as the sunlight intensity during the day and to be zero at night, then this source may be defined by the following periodic function:

$$P_{(x,t)} = P_m \left\{ \frac{2p}{\pi} + \sum_{n=1}^{\infty} b_n \cos\left[2\pi n \left(t - t_s - \frac{p}{2}\right)\right] \right\} \qquad (17)$$

where

P_m = The maximum rate of photosynthetic oxygen production, mg/L.day;

p = fraction of day during which the sun shines;

t = time of day (expressed as fraction of day);

t_s = time of sunrise (expressed as fraction of day);

$b_n = \dfrac{+\pi/p}{(\pi/p)^2 - (2n\pi)^2} \cos(n\pi p)$

For an initial condition of zero deficit, at time of day t = 0 and at a given distance x > 0, i.e., $D_{(x,o)} = 0$, and for an arbitary boundary condition of dissolved oxygen deficit at distance x = 0 and any time of day t, i.e., $D(o, t) = D_o(t)$, Equation 16 has the following analytical solution:

$$\begin{aligned} D_{(x,t)} = {} & D_o e^{-K_a(x/u)} + \frac{K_d L_o}{K_a - K_r}\left[e^{-K_r(x/u)} - e^{-K_a(x/u)}\right] \\ & + \frac{K_n N_o}{K_a - K_n}\left[e^{-K_n(x/u)} - e^{-K_a(x/u)}\right] \\ & + \frac{S}{K_a}\left[1 - e^{-K_a(x/u)}\right] \\ & + \frac{R}{K_a}\left[1 - e^{-K_a(x/u)}\right] \\ & - P_m \left\{ \frac{2p}{\pi K_a}\left[1 - e^{-K_a(x/u)}\right] \right. \\ & + \sum_{n=1}^{\infty} \frac{b_n}{\sqrt{K_a^2 + (2\pi n)^2}} \cos\left[2\pi n\left(t - t_s - \frac{p}{2}\right) - \tan^{-1}\left(\frac{2\pi n}{K_a}\right)\right] \\ & \left. - e^{-K_a(x/u)} \sum_{n=1}^{\infty} \frac{b_n}{\sqrt{K_a^2 + (2\pi n)^2}} \cos\left[2\pi n\left(t - t_s - \frac{p}{2} - \frac{x}{u}\right) - \tan^{-1}\left(\frac{2\pi n}{K_a}\right)\right] \right\} \end{aligned} \qquad (18)$$

The Ministry of the Environment steady state DO model is based on Equation 18. The use of the computer program DOMOD3, based on this model, is described subsequently in the manual.

2.3 Summary of Assumptions

To conclude this section, the following is a review of the assumptions inherent in the above model and the practical limitations to its application.

1) The flow is unidirectional in the x or downstream direction. This assumption is justified in most streams and rivers since the velocities in the lateral (y) and vertical (z) directions are much smaller compared to the velocity in the longitudinal (x) direction.

2)* The streamflow rate is constant in time. The assumption of constant streamflow is appropriate for most flow regimes with the exception of flood flows due to rainfall or snowmelt events.

3) The effluent flow rate is constant in time. This condition is rarely met in practice since there is a diurnal pattern in fluctuation of waste water flows through the treatment facility. However, these fluctuations are generally gradual since they tend to flatten out in the treatment plant. Also, since the waste water is continually discharged into river, the instream processes do not change significantly from day to day. Thus, the assumption of a constant effluent flow on a daily basis appears adequate.

4)* The dispersion in the three principal directions is assumed to be negligible. The assumption of negligible transport by longitudinal dispersion is justified in most cases since it is much smaller in comparison with the transport by longitudinal convection (i.e., transport by the main flow velocity). In most natural streams and rivers the vertical dispersion is complete within a short distance below the waste source. Thus, the assumption of negligible vertical dispersion is also appropriate. However, the assumption of negligible lateral dispersion is not always true since there may exist significant variations in the cross sectional distribution of

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* The segmentation into reaches of a river to be modelled is based on one or more of these points.

concentration for some distance below the source. Under these conditions another model which considers lateral dispersion must be used. For the purpose of the Ministry of the Environment steady-state DO model, lateral dispersion effects are considered by monitoring cross-sectional variations during sampling and estimating a weighted mean value for each parameter.

5)* The temperature is assumed to be constant. Although there is a diurnal variation in temperature, the effect of this variation is somewhat mitigated by the fact that opposing system processes are similarly affected. On the other hand, the seasonal variations in temperature are accounted for through appropriate modifications to the reaction rate coefficients.

6)* All the reaction rate coefficients (i.e., K_r, K_d, K_n and K_a) are assumed to remain constant within a reach. This assumption appears to be satisfactory for most situations.

7)* The sediment oxygen demand rate as well as the algal and plant respiration rate are assumed to be constant throughout the day in a given reach. This assumption is reasonable from a practical point of view.

8)* The biomass of plants and algae remains constant within a given reach (on the order of one day). This assumption is reasonable for short periods, however the biomass may undergo significant changes during the growing season, due to biological factors, or due to significantly changed stream physical conditions.

The photosynthetic activity of plants and algae depends upon many factors such as sunlight, temperature, mass of plants and algae, and nutrients. The effect of all these factors is included in the term, P, which represents the overall rate at which oxygen is released by photo-synthesis. THe photosynthetic process is assumed to vary as the sunlight intensity during the day and to be zero at night. For the sake of simplicity the time variation of the photosynthetic oxygen production is assumed to be represented by a half cycle sine wave which may be described by a Fourier Series. This simplification is valid during days of normal sunlight.

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* The segmentation into reaches of a river to be modelled is based on one or more of these points.

CHAPTER III

WASTE ASSIMILATION STUDY FIELD PROCEDURES

EDITOR'S NOTE

This chapter of the Manual is drawn entirely from "A Manual of Procedures for Conducting a Wastewater Assimilation Study" (1976) by S. M. Irwin and E. T. Smith. It is reproduced here with only minor updating revisions, and a few changes made to ensure consistency with the format of the Manual.

CHAPTER THREE

WASTE ASSIMILATION STUDY FIELD PROCEDURES

3.1 General Strategies

3.1.1 Defining Problem and Objectives

Before any field work is carried out on a stream, the problem(s) should first be defined, and the objective(s) of the study laid out. Preliminary office and field work is often necessary to define the extent of the problem, and determine how the stream reacts to waste loadings before detailed field studies can be planned and carried out.

3.1.2 Preliminary Office Planning

Maps* of the survey area should be obtained, as well as any previous reports on the waterbody and municipal and/or industrial discharges. Municipal and Private Abatement and Industrial Abatement staff can often be of assistance in this task. Pinpoint all discharges to the waterbody on the map.

All existing data on water quality monitoring, streamflow, water takings and consumption, water uses, and volumes and characteristics of waste discharges should be obtained. In addition to MOE staff, local conservation authorities and municipal officials can usually supply a great deal of useful information.

3.1.3 Preliminary Field Studies

If manpower and time permit, the entire reach of stream to be studied should be inspected. Waste discharges, dispersion patterns of effluents and tributaries, physical characteristics of the stream (depth, width, water velocity, type of substrate), water uses, algal growths, the presence of benthic deposits of organic sludges and any other pertinent characteristics should be noted.

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* Throughout this chapter an asterisk (*) sign denotes that the subject indicated is discussed further in another section of the chapter.

One source of information in the field that should not be overlooked is local residents, especially old-timers, who can readily provide information that may otherwise have taken a great deal of time to collect. (e.g. fishery, changes in water quality over the years etc).

Preliminary chemical data (BOD_5, nutrients, day and night-time DO) should be obtained to determine the length of stream to be studied, expected concentrations of chemical parameters, and whether night-time sampling is necessary (i.e. if a diurnal-nocturnal DO variation is evident or if waste loads vary from day to night). This data should be collected far enough in advance of the proposed intensive survey date to allow for the lab analyses to be completed and evaluated.

Based on the preliminary information sampling stations* can be selected. Times of travel (ToT)* should be determined between sampling stations, also noting times to waste inputs and tributary confluences. Observations should be made at three or more different stream flow stages over a suitable range from low flow to average summer flows and data plotted on log-log graph paper (Times of Travel vs Streamflow) which will permit interpolation or limited extrapolation for the streamflow conditions to be modelled.

Mixing studies should be carried out on waste discharges and tributary inflows to determine if lateral and/or vertical multi-point sampling is necessary at any downstream stations were complete mixing of the inflows with the stream water has not occurred.

Stream cross-sections* should be obtained at about 300 m intervals and at those points where the physical character of the stream changes significantly. These data may be used for calculating reaeration coefficients (K_a) of the stream.

At each sampling location, a staff gauge or other stage measuring device (e.g. bench mark) should be installed. Each time the station is visited the gauge height and date should be recorded to develop a gauge height vs flow curve[1] to provide stage height at any flow. This information is necessary to relate cross-sectional data to survey conditions and low flow conditions.

The low flow to be used in the loading designs must be determined or obtained from a hydrologist. The standard design streamflow for continuous discharge to a stream used by the MOE[2] has traditionally been the 20 year, 7 day low flow, 95% chance of exceedence ($7Q_{20}$). Physical data affected by streamflow, such as depth, ToT, mixing, etc., must be extrapolated to this flow. If these physical data are obtained at a flow much greater than the low-flow then extrapolation to low flow conditions is difficult. It is also important to conduct assimilation studies under flow conditions as close as possible to low flow so extrapolation of data to design conditions (low flow) will be minimal.

Where benthic deposits are believed to exert an appreciable demand on the oxygen resources of the stream, benthic respiration* studies should be carried out. The rate thus established will be used as a sink in the dissolved oxygen balance equation.

3.1.4 Pre-Intensive Survey Planning

Based on the preliminary data and the objectives of the study, the type of survey to be conducted will be determined. If 24 hour dissolved oxygen concentrations fluctuate greatly or if wastewater loadings or streamflows fluctutate greatly, then "round-the clock" sampling will be necessary. However, if streamflows and waste loadings are stable and dissolved oxygen concentrations do not fluctuate greatly (i.e. more than 2 mg/l) from day to night then a

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1 Flows may be obtained by streamflow gauging or may be extrapolated by drainage area ratios. See Section 4.3.1.

2 Presently under review; see also Appendix 1.

simple water quality study with limited sampling may be sufficient (enough samples should be taken, however, to provide statistically valid data[1]).

Once the type and scope of the study has been determined, the number of samples that will be submitted to the laboratories can be determined. Parameters that must be measured during an assimilation study are:

- dissolved oxygen (DO) and temperature
- BOD_5
- total phosporus
- soluble reactive phosphorus
- filtered ammonium nitrogen
- total kjeldahl nitrogen
- nitrite nitrogen
- nitrate nitrogen
- BOD_{20}*
- plus any other waste component that may affect water quality and use such as heavy metals or volatile suspended solids (to determine K_r rate if effluent contains high levels of suspended solids).

A tentative survey date must now be selected when streamflow conditions should be low and water temperatures high (mid-July to early September). This date should be verified with all participating personnel and the laboratories. At this time the laboratories will need to know the number of samples expected and parameters requested. WPCP operators and/or industrial contacts must be informed of the expected date of the survey and access to their plants should be arranged for sampling personnel. A contact at this time ensures that the survey is not planned during atypical conditions (i.e. holidays or other plant shut-down periods).

Availability of equipment required for the survey should be checked and sampling bottles ordered from the lab.

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1 A minimum of 12 samples will provide statistically valid data

Survey headquarters should be established, whether it be local MOE office, the local WPCP or a motel. Motel accommodations should be arranged well in advance.

A few days before the survey, the local police departments, the OPP, the local conservation authority and municipal offices should be informed of survey plans. If dye is to be used in the river during the study, these same agencies should be informed. Regional and district MOE staff must also be informed of the survey.

Sample transportation* to the MOE labs must also be arranged.

It is very important that proper maps be prepared with sampling locations and routes clearly marked. A book should be prepared in which all field data will be entered by the sample collector[1].

Staff must be briefed on their duties prior to the survey to ensure they know the route (take them over it prior to the survey), their duties, the sample numbering system* and the work shifts[2].

Finally a check should be made to ensure that all equipment is in proper working order (it is a good idea to have spare equipment available if possible) and that all supplies necessary for the study have been procured (chemicals for Winkler kits, bottles, ice chests, dye, pens and pencils, books, etc).

3.1.5 The Field Survey

The culmination of the preliminary field work and the office planning is a well-executed field survey. This survey generally extends for 30 to 60 hours with samples collected at selected time intervals (generally 3 to 6 hours apart depending on manpower, equipment and time constraints). The duration of the survey depends upon various factors such as:

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1 See Section 3.2.7 - Survey Notes
2 for around-the-clock sampling 12-hour shifts are preferable

a) manpower constraints
b) laboratory capabilities
c) if P & R modelling[1] will be attempted
d) time constraints
e) length of time for sampling run

Enough samples must be collected to provide statistical validity to the data (generally 10 to 12 samples at each station). If discreet sampling cannot be carried out, composite samples may be collected; however, only one value is obtained in this manner and the validity of this one value may be questionable, especially if a bad sample is obtained for any reason (e.g. slug discharge) during the sampling cycle. If P & R modelling is to be attempted enough DO data must be collected to provide a representative diurnal-nocturnal DO curve for about a 30 hour period from before sunrise to after sunrise the following day. The DO model is calibrated for the period sunrise to sunrise but DO data is required at each station prior to sunrise for input to the computer model.

If sampling runs can be conducted every 3 hours, then 12 samples can be collected during a 36-hour period providing statistically valid data and adequate DO data for modelling. The study could be conducted from 9 p.m. one day to 9 a.m. two days hence, to obtain the proper DO curve. If time constraints dictate sampling every 4 hours, then a 48-hour study should be conducted. Timing of the beginning of a 48 hour survey is not too critical as only one complete 30-hour DO cycle can be obtained. However, if data is collected at intervals greater than 4 hours it is difficult to obtain a good diurnal-nocturnal DO graph because the points are too far apart and the high and low DO values for the day may be missed entirely. A 60 hour study with 4 hour sampling intervals will provide 15 samples and if started at 9 p.m. and finished at 9 a.m., 60 hours later, will provide two complete diurnal-nocturnal cycles for modelling needs.

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1 Photosynthesis and Respiration Modelling- for a complete explanation of P & R modelling refer to sections 4.5.6 and 2.2

Figure 3-1 illustrates survey timing to optimize the DO data collected for a 36-hour and a 60-hour study. In any event a survey should not be started or ended at a critical low point on the DO graph (i.e. early morning - 4 a.m. to 8 a.m.).

Situations may arise which may force cancellation or modification of a study. Among them are:

- overcast weather - if survey had been planned to obtain data during hot, dry weather, then overcast weather will change DO curve if diurnal-nocturnal effects of algae are of significance.

- streamflow changes (including rain) - concentrations of parameters change greatly during a rainfall, especially during the first ½ hour of the rain.

- abnormal changes in waste loading - waste loadings are averaged over the survey period; large changes in loads result in changes of concentrations of parameters in the stream during the survey.

These three situations result in changes in the "steady-state" conditions which are desirable for ease of modelling.

Cancellation of surveys because of these conditions may be minimized by:

1) checking with the weather bureau prior to conducting the study. If overcast weather or preciptation is forecast and you do not wish to collect data during an overcast period reschedule the survey. If the survey is partially completed and the skies become overcast, DO levels may be affected and modelling will be difficult. If good DO data has been collected for the required 30 hour period prior to the overcast period then the survey may be ended rather than cancelled or postponed. However, data collected during overcast weather may be valuable as in a stream where there are algal growths, lowest DO levels will occur prior to sunrise of the morning following an overcast day.

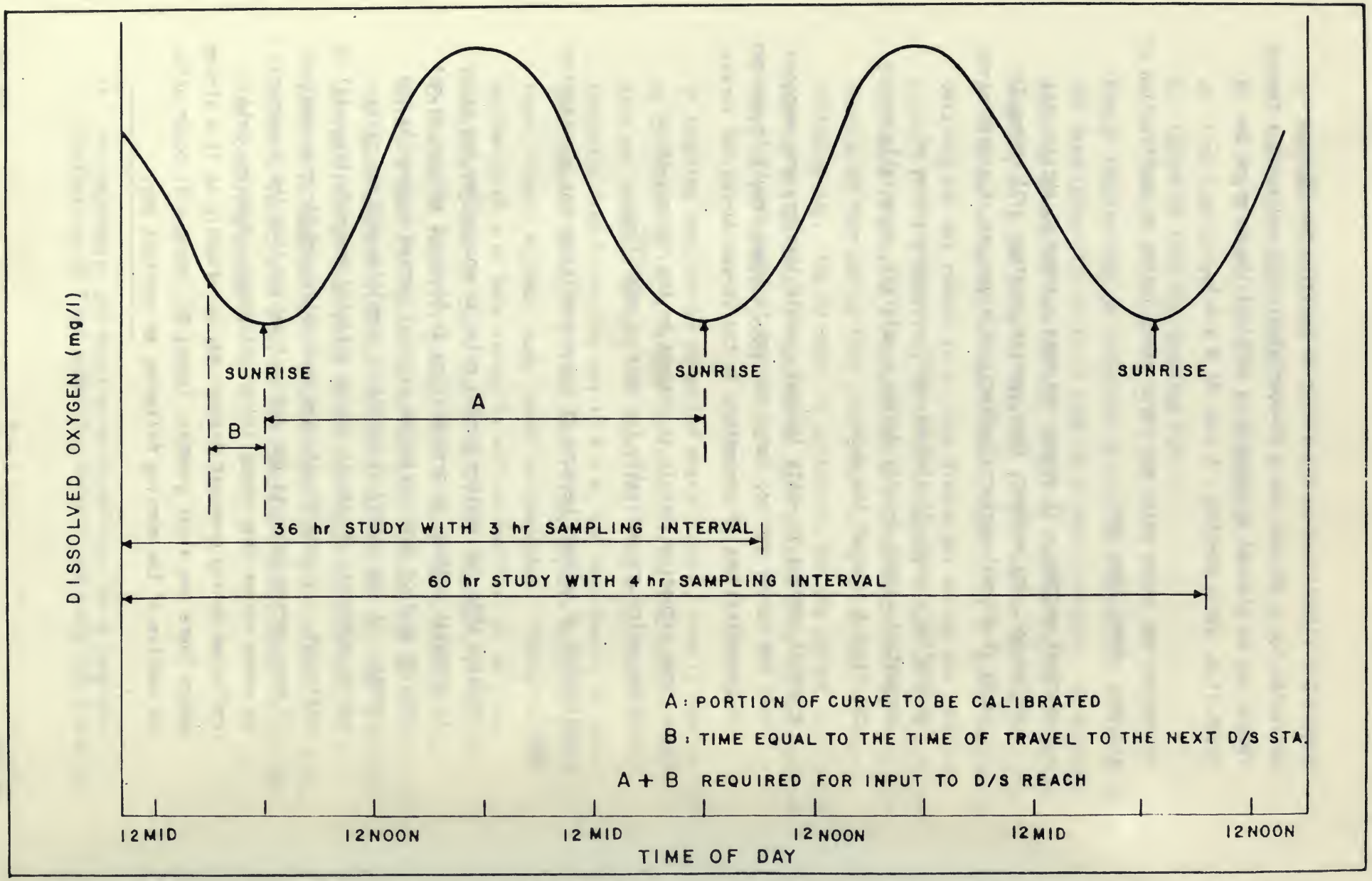

FIGURE 3-1 — STUDY TIMING AS ILLUSTRATED BY TYPICAL DIURNAL NOCTURNAL D. O. CURVE

2) arranging with the proper authorities if streamflow is controlled by dams to ensure steady streamflows for the period of the survey. Rain also changes streamflows and water quality, especially in urbanized areas where storm sewers may discharge large quantities of contaminants into a river during the early portion of a storm.

3) arranging with the WPCP or Industrial Plant operator to attempt to ensure that atypical loading coditions do not prevail during the study, due to construction, plant or equipment breakdown etc.

Duties and manpower required to carry out an assimilation study are:

- day sampling - one man and vehicle per route
- night sampling - two men[1] and 1 vehicle per route[2]
- time of travel[3] - one man and 1 vehicle
- co-ordinator[4] - one man and 1 vehicle
- sample transport[5] - one man and 1 vehicle
- flows[6] by velocity measurement (if necessary) two men and 1 vehicle

Some pointers for carrying out an intensive survey are:

- DO meters should be calibrated to a Winkler titration before each run to ensure accuracy of data. Meters should be calibrated to DO levels in the same range as those expected to be measured.

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1 for safety and ease of working
2 same vehicles as day shift if MOE vehicles
3 may be deleted if graphs have been prepared
4 co-ordinator may perform other tasks if manpower is limited
5 may be carried out by time of travel mean if distance to labs is not too great
6 flows may be taken by time of travel crew (two men)

- recording DO/temperature meters and/or automatic samplers[1] should be installed prior to the start of the survey.

- Care should be taken when installing recording DO meters in the streams to ensure that water is moving past the probe with a velocity of at least 1 cm per second to prevent oxygen depletion at the membrane.

- DO recorders should be calibrated and automatic samplers checked at least once per day[2]. Samples should be removed from the sampler at least once per day as levels of non-conservative parameters may change with time and temperature.

- samples at each location should be collected at least 12 times during the survey (tributaries may not need to be sampled as often). This number of samples should provide statistical reliability to the data.

- DO readings should be taken no more than 4 hours apart to provide a smooth representative DO graph.

- data should be recorded on field data cards (e.g. Figure 3-3 a & b) and any observation out of the ordinary should be noted on the cards (e.g. turbid discharge, rainfall, etc.).

- laboratory submission sheets should be completed at the end of each sampling run to avoid confusion and a back-log of samples with no lab sheets.

- samples should be iced and transported to the laboratory as soon as possible after sampling to ensure accurate data on perishable samples (transport samples at least once per day).

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1 ed. note: vacuum-type automatic samplers have frequently proven unreliable for collecting samples for laboratory analysis (BOD, etc.) and their use is therefore not generally recommended.

2 ed. note: in-stream components of the devices should be checked to see that they are not clogged, fouled with algae, etc.

- time of travel data should be collected during the survey. If graphs of ToT vs flow have been previously drawn up and encompass the flow regime at the time of the study and if manpower is limited, this function may be eliminated.

- bottles may be prelabelled if there are not too many stations or samples to obtain; however, this practice can lead to confusion, especially if the samples are put in the wrong bottles.

- The survey co-ordinator should be available for consultation any time of the day or night during the instensive survey. Portable radios may prove valuable in maintaining communication with crews. Another possible method is to rent a portable paging system i.e. Page Boy (approximate cost $30.00 per month).

- data should be reviewed as it is collected and changes made to plans if the data warrants such changes (e.g. re-locating stations, changing sampling frequency etc.).

3.2 SURVEY PROCEDURES

3.2.1 Maps and Aerial Photographs

Before carrying out any work on the study, the best possible maps of the area should be obtained. Depending on the area of the Province, one or more of the following maps may be available:

topographical - 1:25,000 (1 cm = 250 m)
- 1:50,000 (1 cm = 500 m)
- 1:100,000 (1 cm = 1000 m)

MTC - County - 1:250,000 (1 cm = 2500 m)

MNR Forest Resources Inventory (FRI) Base maps (scale 1:15,840) are very useful for accurate river mileages. They may be obtained from MNR, Forest Resources Group, Whitney Block, Queen's Park, Toronto.

In addition to maps, most of the Province has been covered by aerial photography. A look at the photographs of the area in stereo will sometimes show access points in areas that were previously considered inaccessable (e.g. abandoned roads, trails, farm land-ways, railroad berms etc.).

Physical features of the river valley (e.g. ponds, meandering streams) may also be obtained, facilitating later field operations (e.g. - knowing there is a pond between two stations means a greatly increased time of travel). If aerial photos are not current for that area and if budget permits, a quick low level flight over the area taking your own pictures can be very useful.

3.2.2 Station Selection

Many factors are involved in deciding the location of stations for water quality sampling. The factors include: objectives of the survey, accessibility to the streams, time of travel from waste source, mixing of wastes with stream, physical characteristics of stream (e.g. is location on a sharp bend in the river, upstream or downstream of a dam, in the middle of a large impoundment etc.), length of survey reach, and personnel and equipment availability.

An ideal sampling station is one which would yield the same concentration of parameter no matter where in the cross-section the sample is taken. However, immediately downstream of waste inputs, complete lateral mixing of wastes and stream water rarely exists, so samples should be collected at various locations across the stream. Sometimes vertical mixing does not occur necessitating vertical sampling. Ideally, samples should be collected at various locations and the concentration at each location multiplied by the fraction of flow at each location to give the total loading. Dye dispersion studies can be very helpful in establishing sampling points in a cross-section of stream.

Even in large streams where vertical and lateral mixing are complete, it is good practice to sample at quarter points in the cross-section. Samples should be collected from exactly the same location at each station. Multi-point sampling locations should be marked with paint (if on a bridge) or with poles driven into the streambed (if shallow enough) or with anchored floats in the stream.

Time of travel between stations should be approximately 2 hours (less for small streams, more for larger).

Sampling from the edge of a stream should be avoided. If possible, a rowboat, motor boat or chest waders should be used to sample in midstream (note: always approach the sampling point by walking upstream). However, if a sample must be collected from the river bank, select a location on the outside of a bend where the current flows along the bank. It should be noted, however, that greater dissolved oxygen fluctuations occur along the streambank than in midstream because of the increased aquatic growth and fluctuating water temperature. Suspended solids levels may also be greater along the outside bank because of scouring. Stations should be spaced far enough apart that changes in water quality are measurable, but not too far apart that too great a change in water quality has occurred since the previous station.

All waste sources and tributaries should be sampled where they enter the stream and their volumes should be known. The mainstream should also be sampled immediately upstream of the waste source or tributary to properly assess the effects of that load on water quality. If direct sampling of the waste source or tributary is impossible, then the river should be sampled and gauged directly above the source and at an appropriate distance downstream (immediately downstream for gauging and after complete mixing for sampling). The differences between the downstream and upstream loadings can be attributed to the input.

Control samples (upstream from waste sources) should be obtained for comparative purposes and as inputs to the dissolved oxygen balance equation.

3.2.3 Stream Cross-Sections

Cross-sections of the streambed should be obtained wherever there is a significant change in the physical characteristics of the stream channel.

These profiles are used for calculating the reaeration coefficients (K_a) for each different section of the stream. The data can also be used for calculating times of travel using the volume-displacement method.*

As mentioned in Section 3.1.3 - Preliminary Field Studies, to relate stage height to streamflow for calculating volumes at various flows and to aid in calculating K_a rates at various flows, staff gauges should be installed at stations which are representative of that reach of river. Every time that the station is visited, the stage height should be recorded so a graph can be drawn up relating stage height to streamflow (a log-log relationship).

Cross-sections may be obtained using a graduated pole, a weighted rope or a depth sounder. Stream widths must also be measured (using a tape, graduated rope or stadia and transit). The distances between cross-sections must also be obtained (from a map, by stadia or with a graduated rope).

If stream profiles must be known for flows greater than those under which the cross-sections were obtained, then the profiles should be continued up the streambank to the height required.

3.2.4 Flows

Flow information must be known when carrying out time of travel and dispersion studies, cross-sectioning and intensive sampling (particularly if the stream is to be computer modelled). For modelling purposes it is necessary to know the stream flow, tributary flows and the volumes of all waste inputs.

Streamflows may be obtained from the MOE regional hydrologist for rivers that are gauged by the Water Survey of Canada or the MOE (either by continuously recording or staff gauges). If flow data are not available from this agency, it will be necessary to gauge the stream with a Price-type or Ott current meter[1]. Flow data for the stream and major tributaries should be obtained at least once per day during the survey. WPCP and industrial wastewater flows can normally be obtained from the plant operators (usually in the form of continuous flow chart). Otherwise, temporary weirs, upstream-downstream flow gauging or some other method of measuring the wastewater flows must be implemented.

3.2.5 Time of Travel Studies

Time of travel (ToT) data are a vital part of a water quality study. To determine inputs to the dissolved oxygen (DOMOD3) model, data must be plotted versus time of water passage from the waste input (e.g. K_d rates are calculated from BOD vs to ToT graph, K_n rates from TKN vs ToT graph). Time of travel is an important consideration when selecting sampling station* locations.

Time of travel data will not change unless some physical change is made to, or occurs naturally in the stream (e.g. straightening oxbows, installing dams etc). Varying densities of aquatic weed growths can change ToT values under similar flow conditions. ToT data however, may become useful years later in a follow-up study or if a spill occurs and time of passage to a downstream water supply is needed to determine if protective measures are necessary at a water treatment plant.

Extrapolation of ToT data to low flows may result in erroneous values if data has not been collected over a reasonably large flow range that includes low streamflows.

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1 ed. note: recently, good results have been obtained with electromagnetic current meters such as the Marsh-McBirney Model 711. Consult the appropriate user manual for detailed instructions on the use of any of these meters.

Time of travel may be determined for river reaches by any one of three methods:

a) floats
b) volume-displacement method
c) tracers.

Floats

Surface or weighted floats may be timed from one section to the next to determine time of travel. However, in small streams floats have a tendency to become stuck on tree limbs or other obstacles, to travel into eddy areas or to be moved to the streambank by the river current. They must then be redeposited into the current. Floats are also greatly affected by wind action.

Oranges make very good floats as they move with only a small portion of their surfaces exposed to wind action. They are easily spotted because of their colour, and tend not be entangled because of their spherical shape which allows them to rotate around obstacles.

Because oranges are carried by surface currents that are faster than the average for the stream, the observed time of travel should be multiplied by a factor of about 1.2 to obtain the actual Time of Travel.

Volume - Displacement

A more suitable method of determining time of travel is to measure the cross-sectional area of the stream at frequent intervals, thus determining the volume of the section. A theoretical time of travel can then be calculated by dividing by the streamflow. However, because the whole stream does not move as one mass, the ToT obtained by this method should be verified using dye tracer techniques at a few locations and a factor applied to determine the actual time of travel (usually about 0.8 times the calculated value).

Tracers[1]

a) General:

The most desirable method of determining times of travel is by means of tracers. An industrial discharge may contain an occasional release of some conservative constituent which may be traced downstream by sampling or with a meter (e.g. pH or conductivity). Brine may also be used but because it is heavier than water, proper mixing with the streamwater may not occur or it may simply settle to the bottom of the stream if it is slow moving. Radioisotopes will give good time of travel results, however, safety and public disapproval may preclude their use, especially where public water supplies are taken from the river downstream. Approved fluorescent tracer dyes may be used and can be detected to a level of 3 parts per trillion with a fluorometer.

If an estimate of the time of travel can be made prior to the dye study, using average stream velocity and distance, much time (and possibly dye) can be saved by having a good idea when the dye will reach the downstream location.

Some dye samples can be kept for several days without losing their fluorescence if they are kept away from sunlight (dark garbage bags make suitable storage containers).

Automatic water samplers, timed to sample every 15 minutes are very useful for dye sampling. Their use will free staff for other work and will also permit night-time sampling for dye. Care, however, should be taken in keeping the samples out of sunlight while they are in the sampler.

Dye samples can either be analysed by the MOE laboratories or in the field office using a fluorometer or a spectrophotometer.

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1 ed. note: the use of fluorescent tracers is currently under review to evaluate possible public health implications.

The ideal method of obtaining time of travel data using dye is to proceed approximately a hundred metres upstream from the first section, inject the dye (which has been mixed with water in a container to eliminate density differences), and collect samples at the first and each successive station - about one every minute at the first station starting from before the dye reaches the station until the dye mass is completely past the station. Proceed to the next station and repeat the sampling procedure. Sampling at downstream locations may be carried out at 10 to 15 minute intervals with increased frequency as the dye mass passes the point. The length of time to sample is ideally the length of time between the time of injection and the time the leading edge reaches the station. The time of passage is the time that the centroid (centre of mass) of the dye passes. This centroid may be calculated either graphically, by integration or with a program developed by the Water Modelling Section for the Monroe Statistician 1860 programable calculator. Time of travel is the time required for the passage of the centroid of the dye mass from one station to the next.

In practice however, the dye is normally injected at the first station and samples collected at the next station to obtain the dye concentration vs time graph from which the centroid of the dye mass may be calculated.

Several reaches may be timed using the same dye slug, however, it may happen that sampling at one location might not be finished before sampling must begin at the downstream location, so two crews may be necessary. A form similar to the one presented in Figure 3-2 can be used to record time of travel data.

Many equations have been developed to aid in determining the amount of dye to be injected for a time of travel study. Experimentation is the only way that one can obtain a "feel" for the amount of dye to be used, however, an approximation can be made using the following equation:

$$D = \frac{QLC}{u}$$

TIME OF TRAVEL STUDY – WATER RESOURCES BRANCH

DATE _____ RIVER _____ PARTY _____

DYE INJECTED AT STA. _____ AT _____ am. SAMPLES COLLECTED AT STA. ___

Bottle Number	Time of day	Fluorometer reading & opening				Remarks
		30 x	10 x	3 x	1 x	

GENERAL REMARKS :

TIME OF TRAVEL :
STA. _____ to STA. _____ _____ am pm. to _____ am pm.

TIME OF TRAVEL = _____ hrs. _____ mins.

FIGURE 3-2 – TIME OF TRAVEL RECORD SHEET

where D = dose of dye in ml
Q = streamflow in m^3/sec
L = length of reach in km
C = concentration of dye desired at the end of the reach in ppb
approximately 2 ppb for visual detection)
and u = velocity of stream in m/sec

Remember that too little dye used means wasted effort and too much means a coloured river, perhaps concerning local citizens. If too much dye is used and the fluorometer goes "off-scale", on the least sensitive reading, then dye samples may be diluted with river water before measuring their fluorescence.

An important point to remember when using dye in a stream is that the public will usually see the bright colour near the point of injection and in most cases will telephone some agency to inform them of the "pollution". Consequently, before using any dye in the stream, inform the proper authorities (such as police, local municipalities, conservation authorities etc.) of its use specifying that it is tested for certification as harmless to plants and animals in the concentrations used. Great care should be exercised when using the dye in a stream from which public water supplies are taken downstream.

b) Time of Travel Studies for Large Streams (navigable by boat)[1]:

When a stream is sufficiently large to permit travel by boat, a different technique for ToT studies may be utilized.

In order to eliminate the requirement of having personnel sample for dye at given stations throughout the survey area, a flow-through adaptation of the fluorometer may be utilized. This technique should reduce manpower requirements.

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1 This procedure is a supplement to Chapter 3, contributed by A. Roy and E. Smith of Northeastern Region, Sudbury Regional Office.

Method:

For each survey (different flow conditions) certain key locations are selected for dye injection points. These locations could be waste discharge points, tributary confluences, points of major change in river hydraulics (dam, rapids, etc.), or any other important sampling station. The only requirement that should be kept in mind is that the injection points should be separated by sufficient distance to eliminate the possibility of one dye slug running into another.

At the start of the survey, dye slugs are injected at all key points on the river. After an appropriate time interval (depending on size and mean velocity of the stream) a cruise through the length of the survey area using the flow-through fluorometer will indicate the location of each dye slug. Successive cruises taken during the duration of the survey will track each slug. Every attempt should be made to allow sufficient time for all of the dye slugs to reach the next downstream station where dye was injected.

In addition to these cruises throughout the length of the survey area, time should be available to allow the sampling boat to anchor at specific points of importance.

During successive surveys the dye should always be injected at the same location.

The data generated from this type of survey will require the preparation of an additional set of graphs which will be time vs. river mileage. The origin of these graphs will be time of injection, and mileage point of injection. Each graph will have a line for each individual flow condition. Points may then be taken from this graph to prepare the time of travel vs. flow graph for any specific reach.

3.2.6 Reaeration Coefficient Measurement[1]

Occasionally, the expense and effort required to measure stream reaeration coefficients directly in the field may be justified. A technique which has been used with a fair degree of success in the Ministry is the modified tracer-ethylene gas technique developed by Rathbun, Schultz and Stephens.(1)

This technique is based on the observation that the ratio between the rate coefficient of a gas desorbing from water in a stirred tank and the rate coefficient for oxygen being absorbed by the same water is constant, regardless of mixing conditions. A tracer gas, in this case ethylene, may therefore be injected into a river or stream. Its declining concentration may be noted at downstream sampling stations, and used to estimate reaeration in the section. However, dispersion and dilution effects also influence the concentration changes of the gas and must be accounted for. This is accomplished by injecting a fluorescent tracer dye and assuming dispersion and dilution of the dye is representative of that occuring with the gas.

The coefficient of reaeration (1/day to base e) is calculated, based on the above considerations from:

$$K_a = \frac{1}{R} \cdot \frac{1}{ToT} \cdot \ln \frac{(C_G/C_L)_U}{(C_G/C_L)_D}$$

Where:

C_G = concentration of tracer gas;
C_L = concentration of conservative tracer;
U = subscript denoting upstream end of reach;
D = subscript denoting downstream end of reach;
ToT = time of travel through the reach in days;
R = ratio of desorption coefficient for tracer gas to absorption coefficient for oxygen (in lab studies, has been determined for ethylene-oxygen as 0.89).

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1 This section added as supplemental information to the chapter as originally prepared from S. M. Irwin and E. T. Smith; the technique description is condensed from a report by A. Bacchus.(2)

The equipment required for this type of study includes a regulated tank of ethylene, a high pressure pump with variable rpm, a fluorometer, and special sampling equipment for the ethylene gas containing water samples. The means of injection of the dye is a constant flow Mariotte tank. The ethylene is released from the tank to the stream via a perforated garden sprinkler hose, laid out across the stream and weighted to the bottom.

The complete details of this technique are available from the original source cited above, and from the Ministry's report.(2)

3.2.7 Sample Transportation

Samples for chemical and bacteriological analyses should be transported to the laboratories as soon as possible following sampling; bacteriological samples must be refrigerated and shipped in a cooler if possible. Samples for BOD_5 analysis should be kept cool. Laboratory personnel may request special handling of samples in the field in some cases (e.g. filtering soluble reactive phosphorus samples).

Samples can be transported to the lab by various means including: survey vehicle, train, transport company, air express or freight, bus parcel express (BPX), courier service or, in special cases, private plane. The transportation method employed will depend upon delivery schedules, distance from lab, and costs.

When investigating sample transportation ensure that a schedule of service is obtained and enquire about how long the samples must be at the shipping point before scheduled shipment time. Remember, one minute late for shipment usually means the samples will be a day later arriving at the laboratories. Ensure that there is door-to-door delivery of samples or arrange for their pickup. Let the lab know when the survey is being carried out, so they can be prepared to analyze the samples quickly.

WATER QUALITY STUDY

RIVER BASIN :	SAMPLED BY :
STREAM :	SAMPLING POINT :

LOCATION OF WATER SAMPLING STATION :

SKETCH OF LOCATION :

ANALYSIS :

	5 DAY BOD	NITROGENS	- FREE AMMONIA
SOLIDS	- TOTAL		- TOTAL KJELDAHL
	- SUSPENDED		- NITRITE
	CONDUCTIVITY		- NITRATE
	TURBIDITY		CHLORIDES
PHOSPHORUS	- TOTAL		
	- SOLUBLE		

FIGURE 3-3 a — FIELD DATA CARD (FRONT)

FIELD DATA

DATE OF SAMPLE D M Y	TIME OF DAY	WATER TEMP. °C	D.O. PPM	REMARKS (FLOW, WEATHER, ETC.)

3.2.8 Field Notes

To aid in recording field data in intensive surveys, a separate card should be made up for each station and kept in a note book. Each card should clearly indicate the station number, its location and any special sampling instructions (figure 3-3a). The run number, time of day, DO and temperature and any notable remarks should be entered as the data are collected. (Figure 3-3b)

3.2.9 Sample Numbering

It is very important to clearly label samples to identify the exact time and place the sample was taken.

Station locations should be assigned numbers starting from upstream of the waste source and proceeding downstream through the survey area. It is good practice to label stations prior to the preliminary work and not to change these station numbers after the start of data collection. Any change in numbering could later lead to confusion in applying preliminary data to final data and perhaps might lead to data being applied to the wrong station. If there may be a chance of adding stations later, label the original stations by two's (e.g. 2,4,6 etc.). Stream stations should be preceeded by a letter to identify the stream (e.g. G8 is Station 8 on the Grand River).

To avoid confusion when interpreting data it is good practice to label tributaries with the prefix "T" in sequence with river stations. (e.g. GT39 would be a tributary entering the Grand River between river stations 38 and 40).

WPCP and industrial effluents should be clearly labelled as such so the laboratory will recognize these samples and be able to give them special attention.

If there is more than one sampling point at a location, number the points A, B, C, etc. (or L, C, R) from left to right side of the stream facing upstream.

There are several good methods of labelling bottles in a water quality/assimilation study. One method utilizes station number and time of day. e.g. G18-1418 is Station 18 on the Grand River sampled at time 1418.

Since time of day and run numbers are being recorded in the notebook, another method is to indicate the station number followed by the run number e.g. G18-6 is station 18 sampled on run number 6. The time of run number 6 is recorded in the field note-book. This labelling method is preferred as it is less work for the sampler.

3.3 WATER SAMPLING PROCEDURES

3.3.1 Chemical

Special methods of handling or sample preparation are contained in the MOE Laboratory Service Branch publication "Outlines of Analytical Methods - 1975, MOE." (3)

Manual

Samples should be collected, if possible, directly in the bottle submerged in the river. If sampling by immersing the bottle is not possible then the sample should be collected with a stainless steel, plastic or brass sampling apparatus and transferred to the bottle taking care not to contaminate the sample with the hands or with water from previous sampling. When sampling by hand (i.e. wading into the stream) the sampler should face upstream and grasp the bottle so the water coming into contact with the hands does not enter the bottle, thereby contaminating the sample. The bottle should be immersed below the water surface (except phenol samples which are skimmed from the surface, and samples for ammonium nitrogen analysis, which should not be bubbled) and the water should be allowed to overflow the bottle to ensure a representative sample.

Automatic

Samples may be obtained using an automatic sampler* where manpower is limited or access is poor. Automatic samplers should never be used to collect samples for bacteriological or DO analyses. The sampling hose should be placed in the main current on a stake or rod to obtain a representative sample and to allow the current to continuously cleanse the intake so the build-up of algae will be reduced on the intake. Samples should be removed daily.

3.3.2 Bacteriological

Bacteriological samples must be collected in sterilized bottles. If samples are obtained using a bucket, a bracket to hold the bottle to the outside of the bucket should be utilized. Care should be taken when handling the bottle to avoid contamination of the sample with the hands.

Depth bacteriological samples should be obtained with a depth bacti sampler to avoid contamination from the sampling apparatus.

3.3.3 Dissolved Oxygen (DO)

When taking a water sample for DO analysis, care should be taken to ensure that there is no bubbling of water into the bucket. After the bucket is filled with water let it settle underwater for a few seconds to allow the river water to be exchanged, thereby ensuring that a representative sample is obtained. If possible, obtain the sample for DO analysis directly from the stream using the bottle in the DO field kit.

When performing a wet chemical or Hach DO test, use the appropriate glass stoppered bottle. When taking the sample from a bucket, tilt the bottle as it is slowly lowered into the bucket, allowing the water to flow slowly down the sides of the bottle to avoid bubbling. After the bottle is full of water, hold it under water for a few seconds to allow any small bubbles to escape. Then perform the DO analysis.

TABLE 3-1 - Saturation Values of Dissolved Oxygen in Water at Different Water Temperatures (Under Normal Atmosphere at 760 mm. Pressure)

(Values in mg/L)

Temp (°C)	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	14.62	14.58	14.54	14.50	14.46	14.42	14.39	14.35	14.31	14.27
1	14.23	14.19	14.15	14.11	14.07	14.03	14.00	13.96	13.92	13.88
2	13.84	13.80	13.77	13.73	13.70	13.66	13.62	13.59	13.55	13.52
3	13.48	13.44	13.41	13.38	13.34	13.30	13.27	13.24	13.20	13.16
4	13.13	13.10	13.06	13.03	13.00	12.97	12.93	12.90	12.87	12.83
5	12.80	12.77	12.74	12.70	12.67	12.64	12.61	12.58	12.54	12.51
6	12.48	12.45	12.42	12.39	12.36	12.32	12.29	12.26	12.23	12.20
7	12.17	12.14	12.11	12.08	12.05	12.02	11.99	11.96	11.93	11.90
8	11.87	11.84	11.81	11.79	11.76	11.73	11.70	11.67	11.65	11.62
9	11.59	11.56	11.54	11.51	11.49	11.46	11.43	11.41	11.38	11.36
10	11.33	11.31	11.28	11.25	11.23	11.21	11.18	11.15	11.13	11.11
11	11.08	11.06	11.03	11.00	10.98	10.96	10.93	10.90	10.88	10.86
12	10.83	10.81	10.78	10.76	10.74	10.71	10.69	10.67	10.65	10.62
13	10.60	10.58	10.55	10.53	10.51	10.48	10.46	10.44	10.42	10.39
14	10.37	10.35	10.33	10.30	10.28	10.26	10.24	10.22	10.19	10.17
15	10.15	10.13	10.11	10.09	10.07	10.05	10.03	10.01	9.99	9.97
16	9.95	9.93	9.91	9.89	9.87	9.85	9.82	9.80	9.78	9.76
17	9.74	9.72	9.70	9.68	9.66	9.64	9.62	9.60	9.58	9.56
18	9.54	9.52	9.50	9.48	9.46	9.44	9.43	9.41	9.39	9.37
19	9.35	9.33	9.31	9.30	9.28	9.26	9.24	9.22	9.21	9.19
20	9.17	9.15	9.13	9.12	9.10	9.08	9.06	9.04	9.03	9.01
21	8.99	8.98	8.96	8.94	8.93	8.91	8.89	8.88	8.86	8.85
22	8.83	8.81	8.80	8.78	8.77	8.75	8.74	8.72	8.71	8.69
23	8.68	8.66	8.65	8.63	8.62	8.60	8.59	8.57	8.56	8.54
24	8.53	8.51	8.50	8.48	8.47	8.45	8.44	8.42	8.41	8.39
25	8.38	8.36	8.35	8.33	8.32	8.30	8.28	8.27	8.25	8.24
26	8.22	8.20	8.19	8.17	8.16	8.14	8.13	8.11	8.10	8.08
27	8.07	8.05	8.04	8.02	8.01	7.99	7.98	7.96	7.95	7.93
28	7.92	7.90	7.89	7.87	7.86	7.84	7.83	7.81	7.80	7.78
29	7.77	7.75	7.74	7.73	7.71	7.70	7.69	7.67	7.66	7.64
30	7.63									

When measuring DO with a meter, insert the probe directly into the river or sample bucket and gently agitate the probe (at a velocity of about 1 cm/sec) to prevent oxygen depletion around the membrane. When the readings have stabilized, record the DO and temperature values. Take care to avoid banging the probe on rocks or the sides of the bucket as erratic readings will result.

When installing a DO meter for continuously recording DO, care should be taken to ensure that the probe is situated in the mainstream to avoid oxygen depletion around the membrane and to cleanse the probe. The probe can be suspended on a rod in this stream. Fouling of the probe by floating debris can be minimized by positioning a 30 cm by 30 cm mesh screen about 2.5 m upstream from the probe. This screen must be cleaned periodically.

When calibrating DO meters, 3 Wet Winkler DO determinations should be made and the values averaged. Do not agitate the probe if the meter is being installed for continuous recording of DO/Temperature, as this agitation will not occur in the stream after the installation is complete and erroneously low DO readings will result.

The Wet Winkler method of determining DO gives values in mg/L. Some meters must be calibrated in % saturation.

Table 3-1 shows saturation values of dissolved oxygen at various temperatures. To convert mg/L to % saturation divide the measured DO value by the saturation value taken from Table 3-1 for the appropriate water temperature and multiply by 100.

e.g. Wet Winkler value - 9.6 @ 14.8°C

$$\% \text{ saturation} = \frac{9.6}{10.19} \times 100 = 94.2\%$$

DO readings must be adjusted for differences in elevation (actually differences in atmospheric pressure[1]) according to Table 3-2.

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1 May be obtained from nearby meteorological station e.g. airports

TABLE 3-2 - DISSOLVED OXYGEN CORRECTION FACTORS FOR VARYING ALTITUDES

Atmospheric Pressure (mm Hg)	Equivalent Altitude (m above sea level)	Correction Factor
760	0	1.00
750	100	.99
	200	.98
734	300	.97
	400	.95
716	500	.94
	600	.93
698	700	.92

e.g. if the previous DO value of 94.2% were determined at a elevation of about 300m above sea level, the actual DO reading would be 94.2 x .97 = 91.3%.

3.3.4 BOD_{20} (for k_1 Rate)

A 20 day incubation test is performed to determine the relationship between BOD_5 and the ultimate BOD (BOD_u) of the streamwater. Samples for incubation tests should be taken at the first station downstream from the waste discharge where complete mixing of the waste and streamwater has taken place. If possible, samples for another BOD_{20} test should also be taken in the effluent and also near the downstream end of the survey reach.

A total of 12-1 litre bottles should be taken at each site and all numbered identically. The lab submission sheet should be labelled "For k_1 Analysis - complete nitrogen analyses to be performed on the incubated sample on days 0, 5, 10, 15, and 20 days". The nitrogen analyses are performed concurrently to establish a true k_1 attributable to carbonaceous BOD (CBOD).

Appropriate staff at the lab should be notified prior to submission of the samples to enable them to prepare for the analyses.

3.3.5 BOD_5 with Nitrogens in Parallel

To determine the portion of the BOD_5 (as opposed to ultimate BOD) that is actually carbonaceous, BOD_5 analyses, with complete nitrogen analyses on the first and fifth days, should be taken at least three times at all stations. The difference between the total BOD_5 and the BOD attributable to nitrogens is the carbonaceous BOD_5 (CBOD) which is used to calculate the true CBOD loadings from which a true L_o and K_d can be calculated. It should be stressed that the NOD reaction rate obtained from the above analysis is not the actual NOD reaction rate in the stream but only that which was measured in the bottle. The actual coefficient of nitrogenous deoxygenation (K_n) is determined from the degradation of nitrogens in the stream from station to station and is probably much higher than that which is measured in the bottle.

3.3.6 Special Sampling Techniques and Handling of Samples

"Outlines of Analytical Methods, 1975, MOE" (3) outlines special handling techniques, perishability of samples, preservation techniques special sample containers, volumes of sample required, analytical technique for analysis and special handling of samples.

Input Sampling

One of the most important samples collected during an assimilation study is from the final effluent of an STP or industrial plant. This effluent often contains residual chlorine for disinfection purposes. Chlorine is a strong oxidizing agent which inhibits the BOD test. Although the chlorine may be eliminated by the addition of the proper amount of sodium thiosulphate, the titration procedure is somewhat complicated to perform in the field. It is therefore very important that all samples of final effluents be taken prior to chlorination to ensure a representative BOD_5 concentration.

Bacteriological samples should be taken in bottles specially prepared by the MOE labs for sampling water with chlorine present.

It is wise to obtain a good number of samples[1] of the final effluent, as the BOD_5 and nutrient levels form a very important input to the dissolved oxygen model. It is also a good idea to obtain samples directly upstream from the inputs (including tributaries) and after complete mixing of the stream with the input as a check on your input loading (using mass balance equation[2]).

Sampling For Ammonium Nitrogen

As ammonia gas is released by bubbling action (or converted to other forms of nitrogen) care should be taken to minimize bubbling of the sample for ammonium nitrogen analysis. This may be accomplished by special sampling devices (such as a kemmerer sampler) which can be used to obtain a representative sample below the surface of the water, special pumps which will not bubble the sample, or by carefully obtaining a sample from the surface of the water by allowing the water to gently flow down the sides of the bottle. Wide-mouthed bottles can best be used to sample by this method.

3.4 EQUIPMENT

3.4.1 Water Sampling

Buckets

Sampling buckets should be made from relatively non-contaminating materials such as stainless steel or plastic (e.g. PVC). However, care should be taken when sampling for certain parameters that the bucket material does not contaminate the sample. The sampling bucket should always be rinsed thoroughly before taking the sample.

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1 Twelve samples provide statistical reliable data

2 $C_uQ_u + C_IQ_I = C_DQ_D$
C = concentration
Q = flow
u = upstream
I = Input
D = downstream

Depth Samplers

Various types of samplers can be used to sample water from depths. These are made from brass, PVC or acrylic; some are nickel-plated or teflon-lined to virtually eliminate any chance of sample contamination.

Some depth samplers (such as Kemmerer samplers) are suitable for sampling from bridges. However, care should be taken to reduce the speed of descent of the messenger while above the water's surface (by waving the line) to prevent damage to the sampler, or even its loss by breaking the line as the messenger hits the trigger mechanism.

Automatic Water Samplers

There are many automatic water samplers on the market. Which type of sampler to purchase depends on its application, features and cost. Some of the items to consider when choosing an automatic water sampler are:

- portability (weight)
- power requirements (A.C., D.C. or vacuum operated)
- type of sample obtained - composite - discrete
- type of sample container (glass or plastic)
- amount of sample obtained (is it sufficient for analyses required?)
- is sampling to be proportional to flow?
- can samples be refrigerated within the unit?
- is there any possibility of cross-contamination of samples?
- are materials with which the sample comes in contact corrosion proof and easily cleaned?
- simplicity of operation
- durability
- does it have pre-sample purge cycle to reduce chance of contamination from previous sample and to cleanse the hose end of any acccumulated debris?
- sampling velocity - a too slow sampling velocity allows suspended solids to settle out; a too fast sampling velocity could pick up bottom sediments

3.4.2 DO Meters

Electronic DO meters considerably reduce the manpower required to perform DO tests.

Two different types of electrodes are used in DO meters. The POLAROGRAPHIC type consists of a noble metal cathode and a solid noble metal anode (e.g. YSI Model 54-gold cathode, silver anode).

The other type of electrode is the GALVANIC type with a noble metal cathode and a decomposable anode (e.g. EIL 1510 - silver cathode, lead anode). Membranes are more difficult to change than on the polarographic type. Anodes are consumable and must be replaced about every six months in normal use.

YSI DO meters are automatically temperature compensated, read directly in "mg/L" and are light and easy to transport and store. It is, however, difficult to continuously record data from these meters as the output signal is not compatible with reasonably priced recorders.

The EIL 1510 is bulkier than the YS154 meter, however, a Rustrak Model 288 DC recorder may be plugged directly into the meter facilitating continuous recording of DO and temperature. This meter measures DO in % saturation and the DO readings are temperature compensated.

To save effort and reduce chance of errors, one should consider only temperature compensated models when purchasing DO meters. (Additional features such as salinity compensation are of little value in normal surface water studies in Ontario).

3.4.3 Benthic Respirometer

The Water Resources Branch has developed a respirometer which permits measuring "in situ" oxygen uptake of benthic deposits by passing streamwater over a benthic deposit for several hours. The

oxygen utilized by the benthic deposits is then calculated and expressed in gm O_2 per m^2.day. The benthic oxygen uptake rate (sludge demand) is a dissolved oxygen "sink" in the dissolved oxygen balance equation.[1]

3.4.4 Fluorometer

Approved fluorescent dyes used in time of travel, effluent dispersion and flow measuring studies may be detected in the stream using a fluorometer with filters which measure the appropriate wavelength of fluorescence. Fluorometers are usually portable and may be powered in the field by a small A.C. generator. With a high sensitivity door, the G.K. Turner Model 111 fluorometer can measure fluorescence to about .05 parts per billion (ppb). Optional equipment for this instrument includes a flow-through door, a constant temperature door, and a recorder. A more sophisticated fluorometer, made by Turner Designs, measures fluorescence to about 3 parts per trillion and automatically drops to a more sensitive range when the detection limit of that range is reached. This instrument will operate from a 12 volt wet-cell battery and is a very rugged piece of field equipment. The fluorometer should be calibrated before each use.

3.4.5 Pyranograph

This portable instrument measures and records solar radiation in gm cal/cm^2/min, for either a 24 hour or 7-day period. The recorded data will provide the time of sunrise and sunset and the intensity of sunlight for that day. This information is necessary for input to the dissolved oxygen balance equation when modelling for photosynthesis and respiration.

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1. ed. note: In deeper areas, it may be necessary to take a core sample instead of using the respirometer. In this technique, clear plexiglas tubing is pushed into the sediment and the upper opening capped. The tube is then withdrawn, with an extension rod if necessary, the suction created by the capped end causing the sediment core to be removed with relatively little disturbance. The bottom of the tube is then capped and the sample transported to the lab. Measurements for the benthic oxygen demand are made by periodically measuring DO in the overlying water.

3.4.6 Depth Sounder

This equipment is useful for determining the cross-sectional area of the stream. The data is used in K_a calculations and time of travel calculations in large rivers. Some points to consider when purchasing a depth sounder are:

- chart speed (the faster the better)
- printout - perpendicular printout rather than circular to prevent distortion of recording
- ensure that scale provides adequate profiles at shallow depths
- portability (including the necessary power supply-battery/ generator)
- accuracy
- precision
- durability
- splash proof case

3.4.7 Current Meter

If flows must be obtained manually a suitable current meter such as a Price-type Gurley Current Meter No. 622 or an Ott type C31 should be used. When gauging smaller streams a small cup Pigmy meter should be used with the Gurley Meter.

REFERENCES - CHAPTER 3

1. Rathbun, R. E., Shultz and Stephens; 1975; "Preliminary Experiments with a Modified Tracer Technique for Measuring Stream Reaeration Coefficients"; U.S. Dept. of Interior Geological Survey Open File Report No. 75-256; Bay St. Louis, Miss., U.S.A.

2. Bacchus,A; 1980; "Field Measurements of Stream Reaeration Coefficient"; Water Modelling Section, Water Resources Branch, Ontario Ministry of the Environment; Toronto, Ontario, Canada (Internal report).

3. Anon; 1975; "Outline of Analytical Methods"; Laboratory Services Branch, Ontario Ministry of the Environment; Rexdale, Ontario, Canada.

SELECTED BIBLIOGRAPHY

Anon; 1979; " A Guide to the Collection and Submission of Samples for Laboratory Analysis"; 4th Edition; Laboratory Services Branch, Ontario Ministry of the Environment; Rexdale, Ontario, Canada.

Anon; 1976; Standard Methods for the Examination of Water and Wastewater; 14th Edition; APHA, AWWA, WPCF; Washington, D.C., U.S.A.

Kittrell, F. W.; 1969; "A Practical Guide to Water Quality Studies of Streams"; U.S. Dept. of Interior, FWPCA; Washington, D.C., U.S.A.

CHAPTER IV

THE DOMOD3 DO-BOD STEADY STATE MODEL

CHAPTER FOUR

THE DOMOD3 STEADY STATE MODEL

4.1 Model Description

The MOE steady state DO model, DOMOD3 is based on a modified form of the Streeter-Phelps equation as described in Chapter 2. The assumptions and limitations of the model are summarized at the end of that chapter.

The model provides a mathematical framework which describes the effects of the major processes in a river system on its dissolved oxygen regime. It simulates the decay of CBOD and NBOD input to the system, and the resulting deoxygenation; in addition, the effects on DO of atmospheric reaeration, plant and algal photosynthesis and respiration, and sediment oxygen demand are simulated. The objective of model application is to predict the effects on stream DO of changes in loading rate (from one or more pollutant sources), streamflow, changes in season (where plant effects prevail) and other factors. Observations of model outputs will assist in the determination of management strategies which will result in water quality acceptable by provincial standards.

DOMOD3 requires the following steady state assumptions:

(1) The waste loads entering the stream are constant in time;

(2) the streamflow and temperature are constant;

(3) the instream processes affecting the DO, viz., deoxygenation and reaeration, photosynthesis (except for diurnal variations), etc., remain constant in time.

The above conditions are met in a stream channel of uniform cross-section with a constant waste input and homogeneity of instream processes. However, in natural streams the cross-sectional geometry, as well as the instream processes change from one reach to another. Therefore, the stretch of stream under study is usually

divided into convenient lengths or reaches in which the various parameters affecting the DO regime are considered to be uniform. With such a representation of the system, additional inputs such as tributaries and waste sources can be accounted for.

The following sections describe the data requirements and data analysis required for use of the DOMOD3 model, and instructions on its calibration and application.

4.2 Data Requirements

Details of data required as input to the MOE steady state DO model (DOMOD3) are given in Chapter Three - Waste Assimilation Study Field Procedures.

The data collected during the surveys must be adequate to allow for modelling outside the range of survey conditions. The intensive field studies should be carried out during the period of the year that is the most critical for the stream. For streams receiving continuous waste discharge, the late summer period is appropriate to carry out these studies because, at this time, the water temperature is high and streamflow is low. By carrying out intensive studies under critical conditions, it is possible to evaluate the most probable critical effects of the various forces influencing the stream's dissolved oxygen resources. However, if anaerobic conditions prevail in this period it may be necessary to postpone the survey until aerobic conditions are reestablished in the study stretch of the river.

For streams receiving seasonal discharge during spring and fall, the survey should be carried out during both seasons because effluent quality, stream temperature and flow will vary greatly.

Assimilation studies involve intensive field surveys of 48 to 72 hour duration. They are designed to collect data on streamflows, cross sections, time of travel, dissolved oxygen, temperature and chemical water quality parameters. These surveys allow the monitoring of variabilities in the system and provide data extensive enough to compensate for any data discarded or lost as a result of adverse conditions. Generally, two assimilation surveys should be carried out to provide the data required for (i) calibration and (ii) verification of the model.

The length of stream to be studied is dependent on the characteristics of waste discharged and the time of travel of the system. Generally, municipal waste effluents from secondary treatment plants exert major effects during the first day of time of travel, while the effect of industrial wastes such as pulp and paper mills may extend for a time of travel of five days or longer. The extent of effect depends on the organic loading rate, the biodegradability of the waste, availability of nutrients, presence of toxic materials and the assimilation capacity of receiving stream or river.

The length of the river to be studied should also be based on the length of stream affected by the waste under the highest effluent flow for which the assimilation capacity is to be assessed. Longer distances may have to be studied if predictions are necessary under higher effluent flows or if there is more than one waste discharge.

The data gathered during the preliminary surveys may be used to verify the stream model, if it is not possible to carry out two assimilation surveys (one to calibrate and one to verify the model). This would require two or three chemical sample runs over a 24-hour period including monitoring of dissolved oxygen and temperature at all sampling locations.

4.3 DATA ANALYSIS - HYDRAULIC COMPUTATIONS

4.3.1 Streamflow Estimation:

Daily streamflows should be obtained by direct measurements or from a convenient gauging station whenever physical and chemical surveys are carried out. It is desirable to know flows at both ends of the stream reach every time cross-sectional or time of travel data are collected, particularly on small streams where flow variations can be quite large. Flows of all tributaries and effluents from Water Pollution Control Plants (WPCP) should also be obtained. The collection of these data is very time consuming. To minimize the effort, flows must be obtained at least once a day for major tributaries and at the upstream and downstream ends of the stretch under study. However, flows from WPCP's should be monitored every time effluent samples are collected.

In a typical waste assimilation study measured flow rates will only be available at widely spaced intervals. For intermediate stations, flow data will also be required e.g., to estimate loadings of any parameter at these points. (These loadings can be used to estimate rates of removal, as discussed in section 4.5) The required flow measurements can be made directly, of course, but in many cases this is impractical because of time constraints or shortage of manpower. When this is the case, intermediate station flow rates will have to be approximated.

The following two techniques have been used as a means of approximating intermediate station flow rates, and may both be tried, using one as a check against the other.

(a) Mass Balance

The mass balance method is based on chemical sampling of some conservative substace such as the chloride ion (cl^-) found in the stream and generally found in much higher concentration from the point source (industry, WPCP) at the upper end of the study stretch. The underlying assumption is that at the bottom of any study reach or section, the flux (load) of this parameter is the sum of all the input from upstream:

$$Q_D C_D = Q_U C_U + Q_T C_T + Q_X C_X \qquad (1a)$$

$$Q_D = Q_U + Q_T + Q_X \qquad (1b)$$

Where:

Q = mean flow,
C = mean concentration of the conservative material.

The subscripts indicate the stream locations:

U = upstream station
D = downstream station
X = unknown source
T = intermediate tributary

Equations 1a and 1b may also be expressed in the following working form:

$$Q_U = \frac{Q\,(C_D - C_X) - Q_T(C_T - C_X)}{(C_U - C_X)} \qquad (2a)$$

$$Q_X = Q_D - Q_U - Q_T \qquad (2b)$$

Note that there are 8 unknowns in 2 equations, and that 5 of the unknowns (C_T, C_U, C_D, Q_D and Q_T) will be available from the survey data. Hence the problem reduces to 2 equations in 3 unknowns. An estimate of C_X will therefore provide a solution to 2a and 2b for Q_U and Q_X.

The approach taken is to begin at the most downstream station, estimate C_X for the bottom section and solving the equations, obtain an estimate of Q_U and Q_X. Next, the procedure is repeated, moving upstream one section and using the Q_U from the previous calculation as Q_D for the second section. The procedure is repeated until the most upstream station is reached for which a measured flow is available. If the comparison shows the estimated flow rate to be too high, the estimated C_X used is lowered and the computation repeated until a good agreement is obtained.

There are two ways of initially estimating C_X. The first is to solve for C_X using equations 2a and 2b for the entire study section for which the intermediate flow rates are to be calculated, since Q_U and Q_D are known for these boundary stations. Alternatively, C_X may be estimated from historical chemical data, usually taking the average concentration of the main stream, upstream of any pollutant sources, and from a baseflow period.

This method assumes that C_X is constant for each reach. The accuracy of the estimated intermediate flow rates increases with the accuracy of the chemical data and tributary flow measurements. If $C_X\ Q_X$ is insignificant, accuracy is also improved. The ideal conservative substance would be found only in the main stream, in which case, the accuracy of flow rate estimates would be mainly limited by the accuracy of the chemical data and gauged flow rates.

(b) Prorating of Flows

The basic principle underlying the second method is that the flow produced per unit area from a particular watershed is equivalent to that produced from another watershed of similar geophysical characteristics. To apply this method the drainage area of each reach must be calculated. This is done by marking in watershed boundaries as defined by heights of land on topographical maps, and estimating areas using planimetry.

The drainage areas for all main tributaries for which the flow is known is similarly calculated for all reaches. The difference between the tributary drainage area and the total drainage area for the reach is an estimate of the net contributing area for the unmeasured local flow in the reach and is given by:

$$RDA_I = RDA_{TOT} - \Sigma RDA_T \qquad (3)$$

where:

RDA_{TOT} = Reach drainage area total;

ΣRDA_T = Sum of reach tributary drainage areas;

RDA_I = Net contributing area in reach for local flow

The total unmeasured local flow for the study stretch, Q_I, is calculated by subtracting all known tributary flows and upstream measured flow from measured downstream flow:

$$Q_I = Q_D - Q_T - Q_U \qquad (4)$$

This total local flow can then be allocated to each reach in proportion to the net contributing area for unmeasured flow for that reach:

$$q_i = \frac{RDA_I}{\Sigma RDA} \cdot Q_I \qquad (5)$$

where:

q_i = local fow for the reach;

ΣRDA = the sum of all reach drainage areas contributing to local flow in the study stretch.

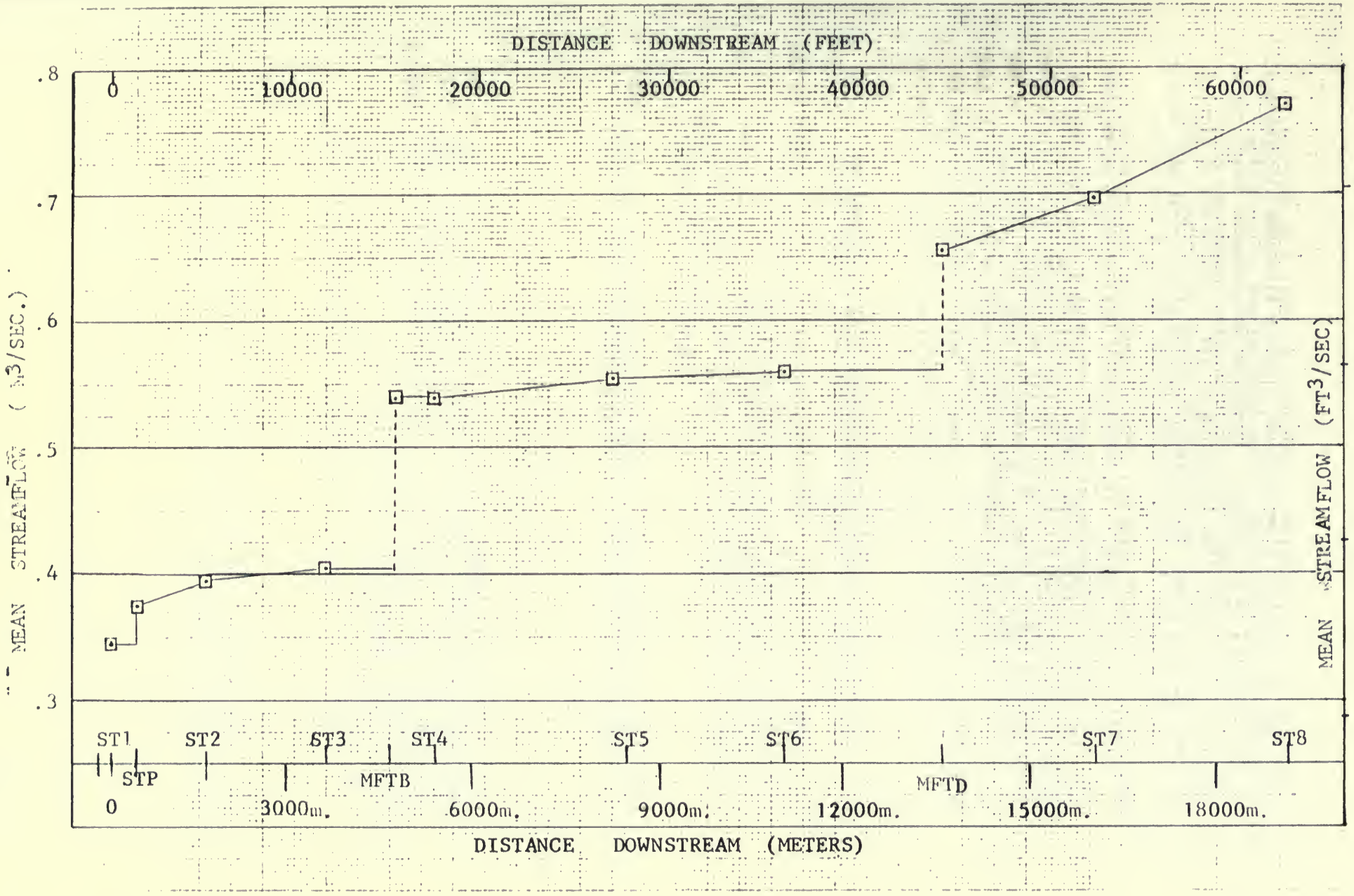


Figure 4-1 MEAN STREAMFLOW AT SURVEY STATIONS VS. DISTANCE DOWNSTREAM SOUTH SAUGEEN RIVER AUG. 28-31, 1973

Figure 4-1 is a plot of flow vs stream distance estimated using method (a) for the Mount Forest survey. Streamflow measurements were taken at Station 1, first concession downstream of STN. 3, MFTB, and at Stn. 7, and the flow of the WPCP was obtained from the plant operational records.

If the mean flow of the WPCP is subtracted from all stations downstream from it, ratios of streamflow to flow at the head of the study section can be calculated at each station. By applying these ratios to any flow at the head of the reach occuring under the same basin conditions as the survey flow and adding the WPCP flow, the streamflow estimations at each survey station are made for all surveys. These flows are then used to aid in the estimation of any parameter such as width, depth and velocity.

4.3.2 Relationship of Depth, Width and Velocity to Streamflow

The streamflow (Q) at any stream cross-section is directly related to the cross-sectional area (A) and mean velocity (U) by:

$$Q = AU \tag{6}$$

The area of any cross-section is equal to the product of the mean depth (H), the mean top width (W), and the mean velocity (U).

$$Q = H \times U \times W \tag{7}$$

This concept can be extended to an entire river reach: that is, we can establish a mean depth, mean width and mean velocity for a reach. The mean velocity can be found from the measured time of travel (T) through a reach and the length of the reach (x); i.e.

$$U = \frac{x}{T} \tag{8}$$

Mean width and depth are found by measuring a suitable number of cross-sectional profiles in a stream reach (see section 4.3.4).

The depth of a river is often highly variable both in cross-section and longitudinally, whereas width may be less variable. Accordingly, mean depth can also be calculated by solving equation 7 for depth, H, when the flow, velocity and average width are known. Some sources state that this procedure is in fact the preferred method of calculating depth, since it maintains the relationships expressed below in equations 9 through 12.

When the streamflow (Q) is changed, the width (W), depth (H), and velocity (U) or time of travel (T) will be affected. The following general relationships, as developed by Leopold and Maddock may be used to derive the new H, U, T and W:

$$H = aQ^{d} \quad (9)$$

$$U = bQ^{f} \quad (10)$$

$$T = b_{t}Q^{-f} \quad (11)$$

$$W = cQ^{g} \quad (12)$$

Where a, b, b_t and c are empirical constants, and d, f and g are exponents which are functions of the hydraulic radius, slope and roughness of the channel.

It can be shown by substituting equations 9, 10, 12 in equation 7 that:

$$abc = 1 \quad (13)$$

$$d+f+g = 1 \quad (14)$$

These equations provide us with a hydraulic model requiring only easily determined empirical exponents (d, f and g).[1] They are extremely useful to estimate the width, depth and velocity of a reach under flow conditions not surveyed. The depth and velocity of a stream control its reaeration, and hence its assimilative capacity at a given flow.

---

1 See sections 4.3.3 and 4.3.4

The Leopold-Maddock equations can be used to modify a model parameter for a change in streamflow that is directly dependent on one or more of the relationships (i.e., between H, U or W and Q). For example, the photosynthesis parameter P_{av} is inversely proportional to depth[1] and hence,

$$\frac{P_{av_2}}{P_{av_1}} = \frac{H_1}{H_2} \qquad (15)$$

From equation 9, the effect of a change in Q on P_{av} can be determined as follows:

$$\frac{H_1}{H_2} = \frac{a Q_1^{\,d}}{a Q_2^{\,d}} = \left(\frac{Q_1}{Q_2}\right)^d \qquad (16)$$

By equating equation 15 and equation 16 and rearranging, we obtain the following:

$$P_{av_2} = P_{av_1}\left(\frac{Q_1}{Q_2}\right)^d \qquad (17)$$

The Manning equation provides another hydraulic model that requires a knowledge of three of four parameters; these are velocity (U), slope (S), hydraulic radius (R) and roughness (n) of the channel:

$$U = \frac{R^{2/3} S^{1/2}}{n} \qquad (18)$$

The Manning equation is accepted as more accurate than the corresponding Leopold-Maddock equation (equation 10) in hydraulic engineering; in water quality engineering, however, the latter equation is more commonly used because of its simplicity.

---

1 assuming the areal photosynthetic rate remains constant

Attempts to use the Manning equation for whole reaches in shallow streams have met with little success. This lack of success probably reflects our inability to determine the wetted perimeter when rocky, or debris, or plant covered stream bottoms are encountered. In addition, stream cross-sections, slopes and velocities are rarely uniform for any appreciable length of stream. Any error is generally assigned to the roughness parameter, n. Since this parameter is assumed to remain constant throughout the stretch of stream under study, large errors in predicted depths or velocity may result if there has been an error in estimating n. This limits the use of the Manning equation for our purposes; however, if data on the channel geometry and bottom roughness are available, the Manning equation could be used instead of the Leopold-Maddock equation.

In the absence of data on a stream under study, Leopold-Maddock coefficients can be derived by referring to the Manning equation. For a rectangular channel, the Manning equation gives $f = 0.4$, $d = 0.6$ and $g = 0$.

4.3.3 Time of Travel and Velocity

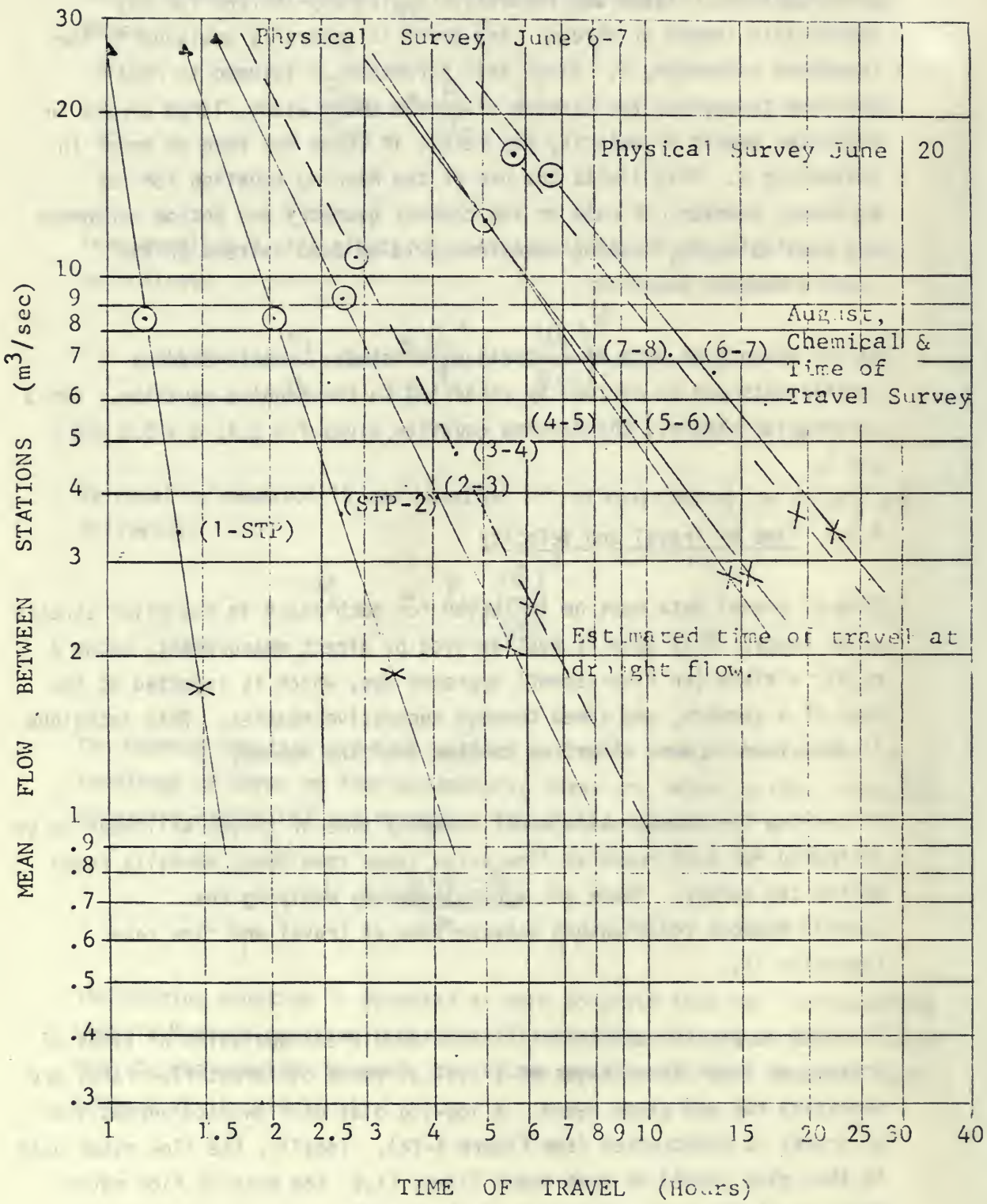
Time of travel data must be collected for each reach in the river stretch under study. This data is best derived by direct measurement, using a highly visible (or fluorescent) approved dye, which is injected at the head of a stretch, and timed through successive reaches. This technique is described in more detail in Chapter 3 of the manual.

In running the steady-state model, usually time of travel will have to be estimated for each reach at flow rates lower than those normally found during the survey. These are approximated by applying the Leopold-Maddock relationship between time of travel and flow rate (equation 11).

In order to provide sufficient data to enable extrapolation of times of travel, at least three times of travel at three different flow rates are necessary for any given reach. A log-log plot of flow rate versus time of travel is constructed (see Figure 4-2a). Ideally, the flow rates used in this plot should be mean reach flows; (i.e. the mean of flow rates measured at the head and bottom of the reach).

Fig. 4-2a

SOUTH SAUGEEN MOUNT FOREST
FLOW VS. T E of TRAVEL
SUMM 1973



To estimate flows, the mean survey flow is used, (i.e. the mean of the daily flows recorded at the head of the first reach during the study). For successive reaches downstream, one of the techniques described in section 4.3.1 is used to add incremental flow rate increases. Using this technique, some problem may be encountered if sources such as a WPCP are present and undergo diurnal changes in discharge rate while comprising a significant portion of the downstream flow. In this case, flows coincident with the time of travel measurement should be adjusted by referring to the flow records of the WPCP.

Large changes in the biomass of aquatic plants and algae as well as over-bank flows may also cause non-linearity in the plots. Generally, time of travel studies should be confined to flow regimes that have within bank flows as it is this period with which management is most concerned. Variable plant mass is a problem more difficult to compensate for; it could best be dealt with by noting areal coverage of biomass present at the time of study and undertaking further time of travel measurements under similar coverage conditions; this, however, is usually not possible.

To extrapolate the relationship between time of travel and flow rate to other flowrates, the linear plot, as described above, can be used. Alternatively, the Leopold-Maddock relationship (equation 11) on which the linear plot is based can be used. As stated earlier, equation 11 has the following form:

$$T = b_t Q^{-f} \qquad (11)$$

For computational purposes, the following form of the equation is more useful:

$$\frac{T_2}{T_1} = \left(\frac{Q_2}{Q_1}\right)^{-f} \qquad (11a)$$

The value of f in equation 11a is determined from the slope of the plotted line which best fits the survey data. The slope is calculated by using the following equation:

$$slope = -f = \frac{\log (T_2/T_1)}{\log (Q_2/Q_1)} \qquad (19)$$

where (T_1, Q_1) and (T_2, Q_2) are any pair of points on the line.

The value of b_t may be found by solving equation 11 for known values of T, Q and -f. Note that the slope of the plotted line is the negative of the exponent in equation 10. This follows since time of travel is the reciprocal of velocity in a particular reach. Also, equation 10 and 11 can be related since $b_t = \frac{x}{b}$.

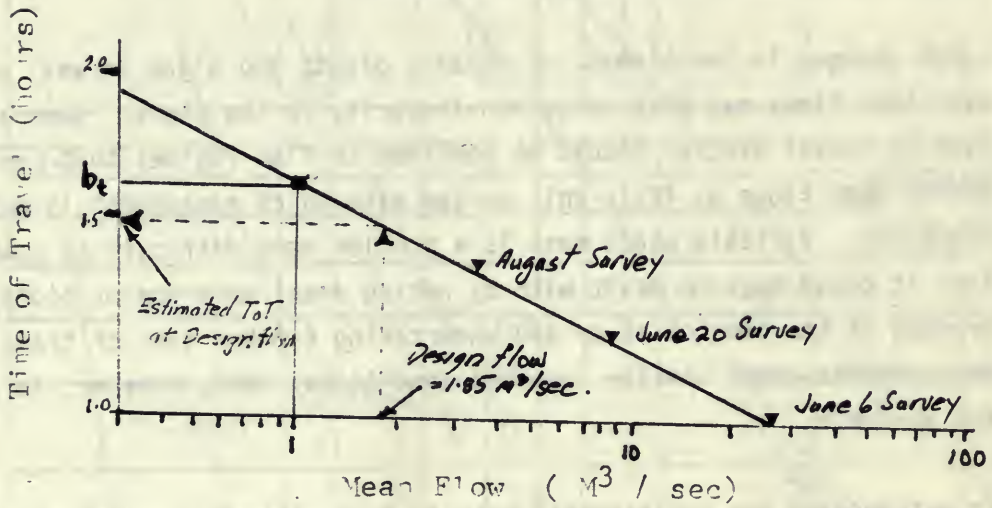


Figure 4-2b
Estimation of Time of Travel Using Graphical Method

Example: The time of travel data for reach (1-STP) has been replotted from figure 4.2a, and is shown above. Determine the time of travel for a design flowrate of 1.85 m^3/sec.

Solution: Calculate the slope of the plotted data using equation 19.

Choosing a pair of points on the line:

Q_1 = 1.8 m^3/sec
Q_2 = 28.0 m^3/sec
T_1 = 1.5 hr.
T_2 = 1.0 hr.

therefore: $\text{slope} = \frac{\log (1.0/1.5)}{\log (28.0/1.8)} = .147$

therefore: $f = -(-.147) = .147$

From equation 11: $b_t = \frac{T}{Q^{-f}}$

Substituting in one pair of data, Q, and T_1 from above:

$$b_t = \frac{1.5}{1.8^{-.147}} = 1.64$$

This value is indicated in figure 4.2b. Note that b_t may also be read directly from the plot as T when Q = 1; i.e., $b_t = \frac{T}{1^{-f}} = T$

Hence the time of travel for the design flow of 1.85 m^3/sec is:

$$T = b_t Q^{-f}$$
$$= 1.64\ (1.85)^{-.147}$$
$$= 1.49 \text{ hours}$$

Alternatively, the time of travel may be read directly from the plot, as shown by the dotted line in Figure 4.2b.

4.3.4 Cross-Section Data - Determination of H and W

For the purposes of the following sections of the manual, some terms must be defined in order to clarify references to distance along a river or stream under study: (1) A reach is a length of stream or river between two designated stations. (2) A section is a length of stream or river, several of which comprise a reach, and which may lie entirely between two points designated for taking cross-section measurements. Alternatively, these measurement points may lie within a section of stream such that cross-sectional measurements at the point are generally representative of that section. (3) A stretch is a length of stream or river comprised of one or more reaches.

It is essential to derive an accurate set of measurements on stream channel morphometry in any detailed assessment study. A set of cross-sections of the stream channel needs to be derived in order to establish mean depth, H, and mean top width, W for each study reach. In turn, these parameters are applied to derive estimates of reaeration capacity; relationships between mean reach width and flow, and between mean reach depth and flow also need to be established.

The first step in this procedure is to establish several cross-section locations between main stations. Reach by reach determination of H and W will be accurate in proportion to the number of cross-sections measured and the channel uniformity between cross-sections. Since time constraints usually limit the number of cross-sections which may be surveyed, the approach is to select cross-section locations which are judged to best represent the contiguous channel sections. The sections should be defined during the preliminary survey based on observations of channel characteristics which differentiate adjacent sections.

The next step in the procedure is to plot all cross-section locations on a master map of the survey area. All distances between stations are then obtained from field sheets, if the distances have been measured, or scaled from the master map. Cumulative distances to each cross-section are then calculated. As an example of these procedures figure 4-3, shows a map of cross-section stations used for the S. Saugeen R., and table 4-1 summarizes data from these stations.

Each field cross-section should be plotted on a squared paper cross-section sheet, as shown in figure 4-4. The area of the cross-section, for any particular water level may be determined by counting the squares bounded by the water surface and channel bottom, and multiplying by the square constant. The average depth is found by dividing the area by the width at the water line.

One of the objectives of the cross-sectional data analysis is to establish a plotted relationship between mean reach width (or depth) and flowrate. An example plot is shown in figure 4-5. For this purpose, the flowrate and cross-sectional measurements must be known on at least three occasions having different flowrates.

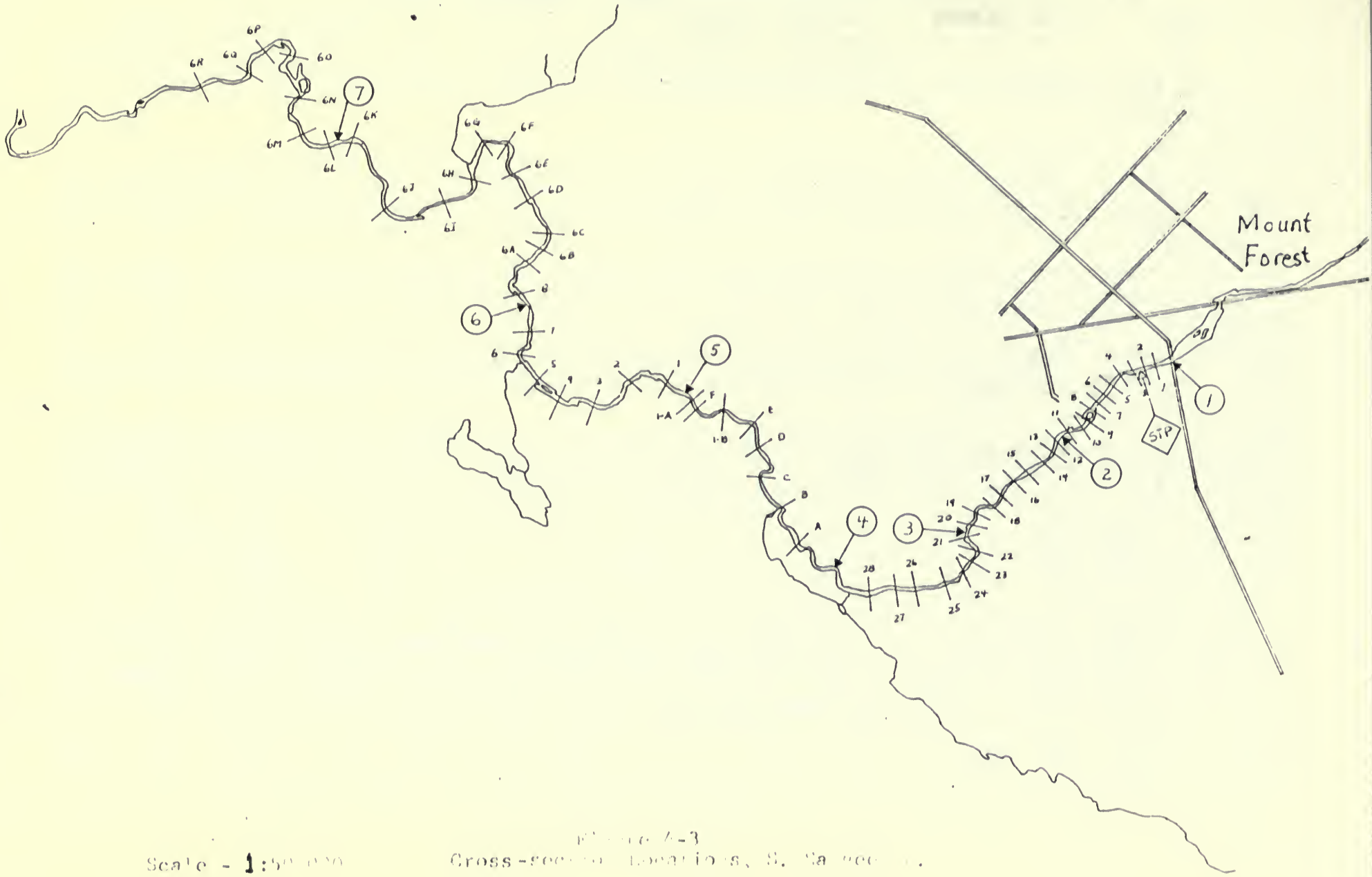


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Figure 4-3
Cross-section Locations, S. Saugeen R.

Table 4-1

Cross-section Data Summary

S. Saimeen R. Physical Survey

Station	Section Number	Date D/M/Y	Mean Depth (m.)	Width (m.)	Area (m^2)	Section Length (m.)	Distance Down-stream (m.)	Flow (m^3/s)
1	1	06/6/73	0.235	16.2	3.81	198.1	198.1	2.55
	2		0.244	12.5	3.05	167.6	365.7	
	3		0.185	13.7	2.53	137.2	502.9	
STP	4		0.159	22.3	3.55	143.3	646.2	
	5		0.171	18.3	3.13	160.0	806.2	
	6		0.327	17.7	5.79	137.2	943.4	
	7		0.320	11.6	3.71	137.2	1080.6	
	8		0.649	14.6	9.48	137.2	1217.8	
	9		1.061	18.9	20.05	137.2	1355.0	
	10		0.238	22.6	5.38	143.3	1498.3	
2	11		0.093	17.1	1.59	182.9	1681.2	
	12		0.144	15.5	2.23	137.2	1818.4	
	13		0.232	21.3	4.94	137.2	1955.6	
	14		0.320	13.7	4.38	213.3	2168.9	
	15		0.692	19.8	13.70	213.3	2382.2	
	16		0.503	22.6	11.37	228.6	2610.8	
	17		0.320	15.2	4.86	228.6	2839.4	
	18		0.419	17.4	7.29	228.6	3068.0	
	19		0.253	18.0	4.55	221.0	3289.0	
	20		0.448	17.4	7.80	213.3	3502.3	
3	21		0.343	14.0	4.80	243.8	3746.1	
	22		0.265	17.4	4.61	213.3	3959.4	
	23		0.384	17.1	6.57	213.3	4172.7	
	24		0.491	11.9	5.84	213.3	4386.0	
	25		0.512	18.3	9.37	213.3	4599.3	
	26		0.195	21.0	4.10	259.1	4858.4	
	27		0.573	16.8	9.63	213.3	5071.7	
4	28		0.844	17.4	14.69	274.3	5346.0	2.55
	A	20/6/73	0.411	9.1	3.74	1014.4	6360.4	0.85
	B		0.354	9.1	3.22	482.8	6843.2	
	C		0.305	10.7	3.26	354.2	7197.4	
	D		0.122	15.8	1.93	370.0	7567.4	
	E		0.305	14.9	4.54	289.6	7857.0	
	F		0.223	12.8	2.85	406.6	8263.6	0.85
	1-A	08/6/73	0.305	20.4	6.22	221.0	8484.6	2.55
5	1-B		0.410	19.8	8.12	22.9	8507.5	2.55
	1	21/6/73	0.199	10.7	2.13	331.3	8838.8	0.85
	2		0.194	17.7	3.43	563.3	9402.1	
	3		0.445	19.5	8.68	386.2	9788.3	
	4		0.226	14.3	3.23	338.0	10126.3	
	5		0.198	13.7	2.71	305.7	10432.0	
	6		0.234	8.8	2.06	338.0	10770.0	
6	7		0.143	9.1	1.30	338.0	11108.0	0.85

Distance Between Cross Sections (metres)

1681.2

2064.9

1599.9

3161.5

2600.5

Figure 4-4

CROSS SECTION SHEET

RIVER: South Saugeen
LOCATION: B
DATE: 20/6/73

WIDTH: 11.7 m. AVERAGE DEPTH: 0.425 m.
AREA: 4.53 m^2.
STREAM FLOW: 1.27 m^3/sec
DISTANCE FROM PREVIOUS CROSS-SECTION: 482.8 m.

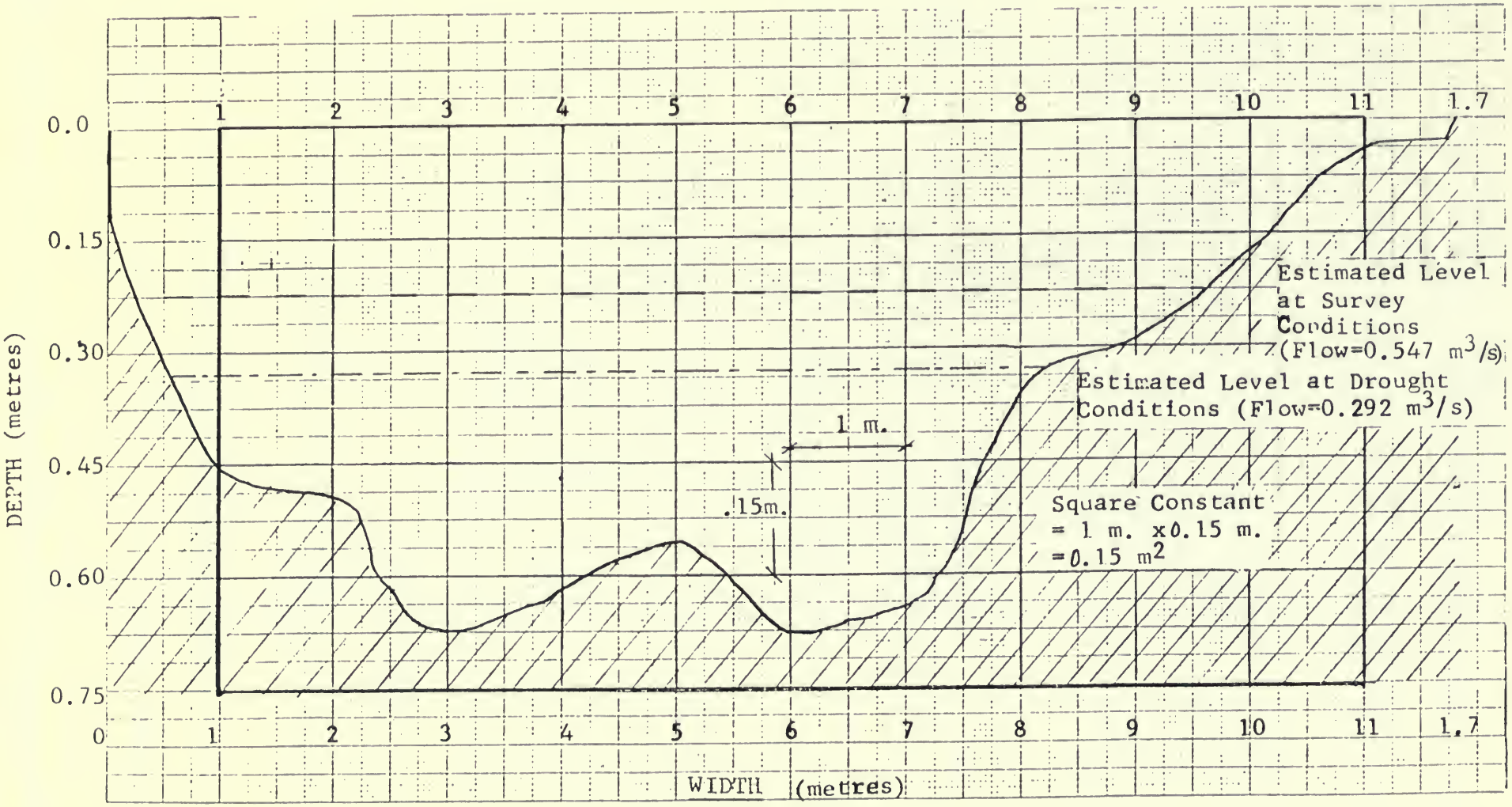


Figure 4-5a

Log-Log Plot of Streamflow (m^3/sec)
vs River Depth (m)
South Saskatchewan R.

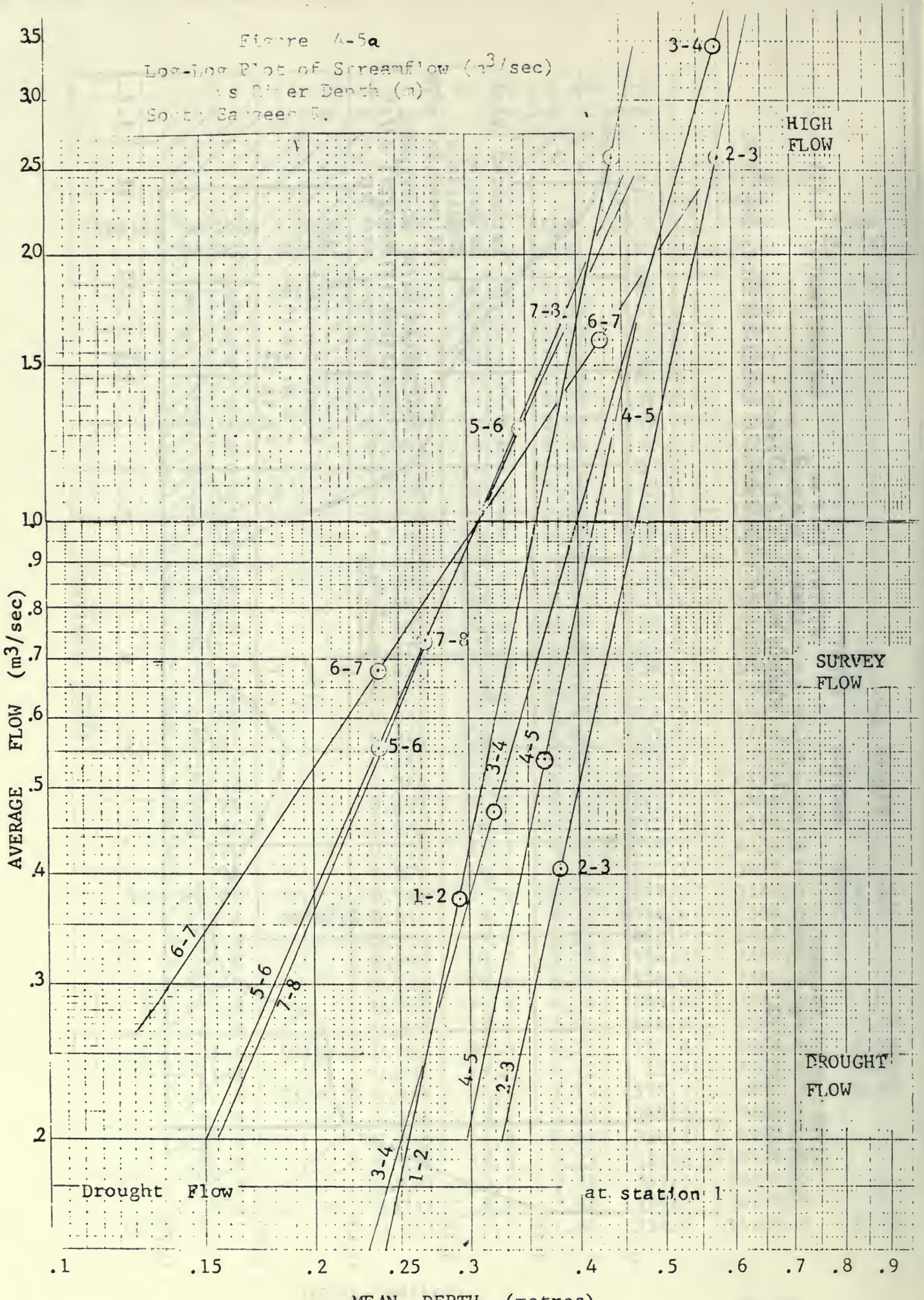
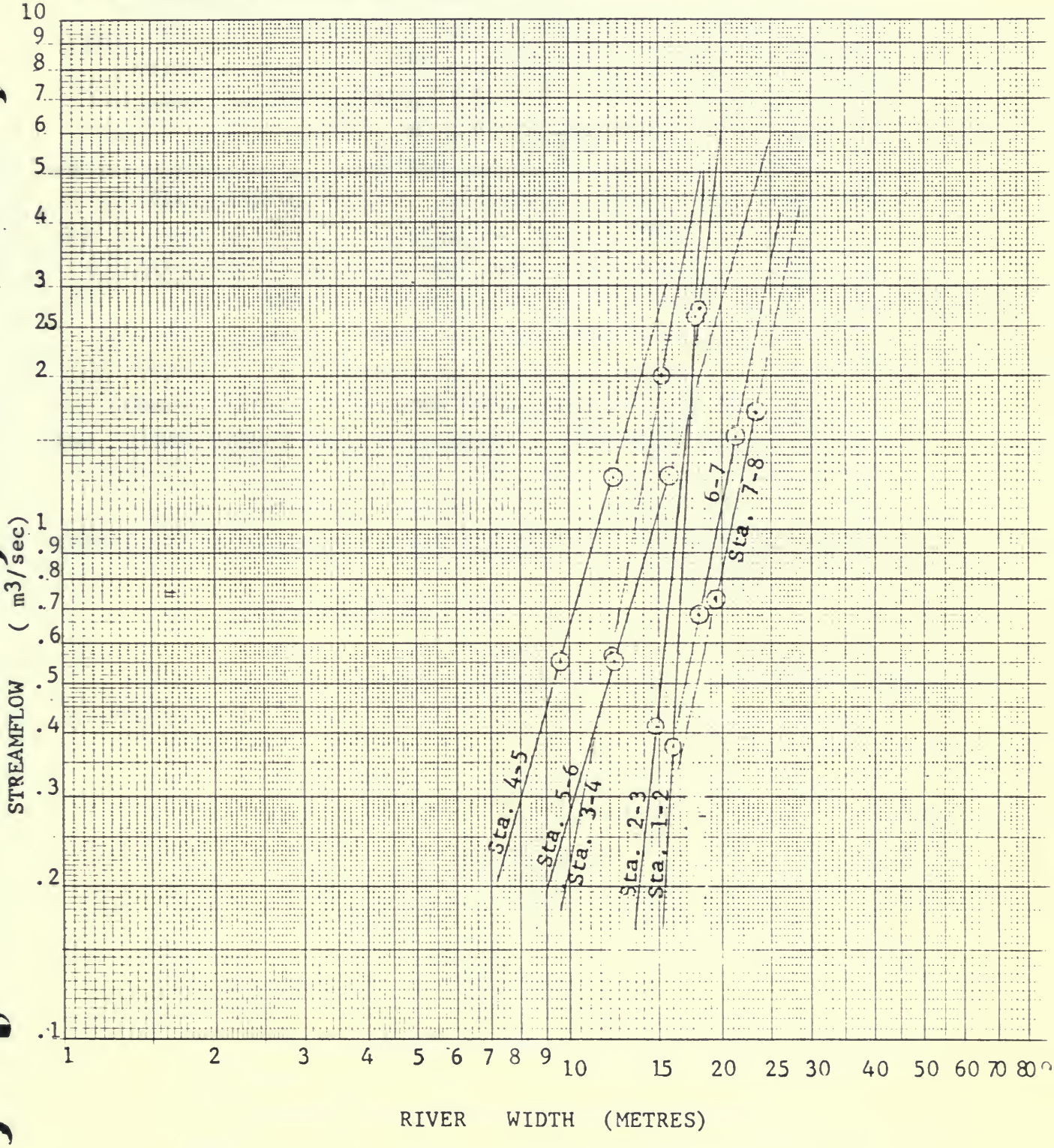


Figure - 4-5

LOG-LOG PLOT OF STREAMFLOW (m^3/sec) VS.
RIVER WIDTH (m) OF THE SOUTH SAUGEEN RIVER, MOUNT FOREST, ONTARIO.
(Flow between stations as shown)



It would be excessively costly to re-measure water levels at each cross-section at each different flowrate; however it is unnecessary to do so if reference levels have been provided by establishing staff gauges or marked stakes at key points throughout the study reaches. The new cross-section water levels can then be estimated from the changes in water levels on the stakes or staff gauges. This is done by converting the change in water level at the measurement point to a percentage of the mean depth of the stream at that point. This percentage can then be multiplied by the mean depth of the appropriate cross-sections to obtain estimates of change in water level. The widths under the new flowrate condition are scaled from the cross-section plots, after marking in the new water levels, and the new area calculated as previously described.

The mean width and depth calculations for sections can now be worked up to the level of stream reach. If cross-section measurement stations were nearly equally spaced, the mean reach width and depth will be a simple mean of the values derived for the sections in the reach. However if there is variability in the section lengths, a mean weighted by section length is necessary.

e.g.

$$H_r = \frac{\left(\sum_{i=1}^{N} d_i H_i\right)}{L} \qquad (20)$$

where:

H_r		=	mean reach depth;
h_i	$(i = 1,N)$	=	section mean depth;
d_i	$(i = 1,N)$	=	section length;
L		=	$d_1+d_2+d_3 + \ldots + d_N)$ = reach length;
N		=	number of sections.

Once the three mean reach width and mean depth points have been calculated, the graphs are plotted (see Figure 4-5). From these the exponents d and g in equation 9 and 12 are determined as for f determined in equation 11, in section 4.3.3. The sum of d, g, and f is then compared to 1. The difference between the sum of the exponents and 1 should be less than 10 percent. Any error found can generally be assigned to the depth and width exponents unless it is an error introduced by streamflow, since the time of travel values are measured directly.

If the error is greater than 10%, with the widths and depths taken from the cross-sections, the parameters should be re-evaluated. If the sum of the exponents is greater than 1 then the change in water surface elevation for flow should be reduced and the depth and width values re-estimated. If the error arises because of the uncertainties in the streamflows then the flows used for plotting may need to be changed.

4.4 Evaluation of Model Parameters

4.4.1 Biochemical Oxygen Demand

The steady state DO model is designed to allow for point source inputs of two types of oxygen-demanding substances: carbonaceous BOD and nitrogenous BOD (CBOD and NOD). The distinction is made because of the different rates at which these substances exert their effect on the dissolved oxygen regime of the receiving water. A further distinction should also be made between ultimate carbonaceous BOD ($CBOD_u$) and the five-day BOD (BOD_5). Water samples taken at most stations will usually be analysed for BOD_5, which is the most convenient measure of the strength of the effluent at that location; however, the $CBOD_u$ should also be known, for the following reasons:

(a) to determine the effects on receiving water when retention time in the system exceeds 5 days;
(b) to characterize the potential biochemical degradability of the waste specific to the location; the rate at which degradation proceeds may vary over time, quite differently for two different effluents.

This section presents two selected methods for determining the $CBOD_u$ from long-term BOD analyses and for using the results to derive the $CBOD_u/CBOD_5$ ratio; this ratio is used to convert the $CBOD_5$ results obtained during a study, to $CBOD_u$ values.

4.4.2 $CBOD_u$ Determination

The reader is referred to Chapter 3 for guidelines on the collection and submission of samples for BOD determination. The laboratory procedure consists of incubating a sample at 20^oC and determining its oxygen uptake daily until all biochemical oxygen demand has ceased.

The numbers of nitrifying bacteria in the original sample can greatly affect the interpretation of this test in arriving at a determination of $CBOD_u$. Figure 4-6 shows a result typical of the case where low levels of nitrifiers were present in the original sample. In this figure, an increased rate of oxygen uptake is evident after about 8 days, and this latter "second-stage" curve is largely attributable to the activity of nitrifying bacteria. In estimating the carbonaceous component of the BOD, the curve of oxygen uptake before the onset of nitrification ("first-stage" demand) should be extrapolated, as indicated by the dotted line in the figure.

By contrast, samples originating from water receiving effluent which has an actively nitrifying bacterial population may not exhibit the two distinct stages of oxygen demand; such samples may be associated with water receiving municipal sewage treatment plant effluent. Where this condition is expected, the laboratory performing BOD analyses should be requested to use chemical inhibition of nitrification. An example of the effect of the use of ATU as an inhibitor is shown in Figure 4-7.

Figure 4-

TWO-STAGE BOD CURVE

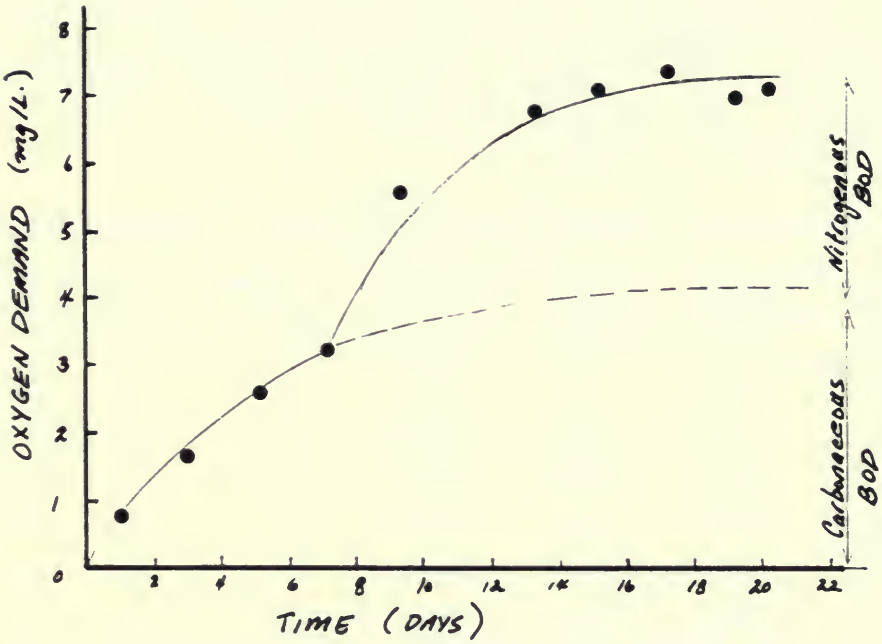
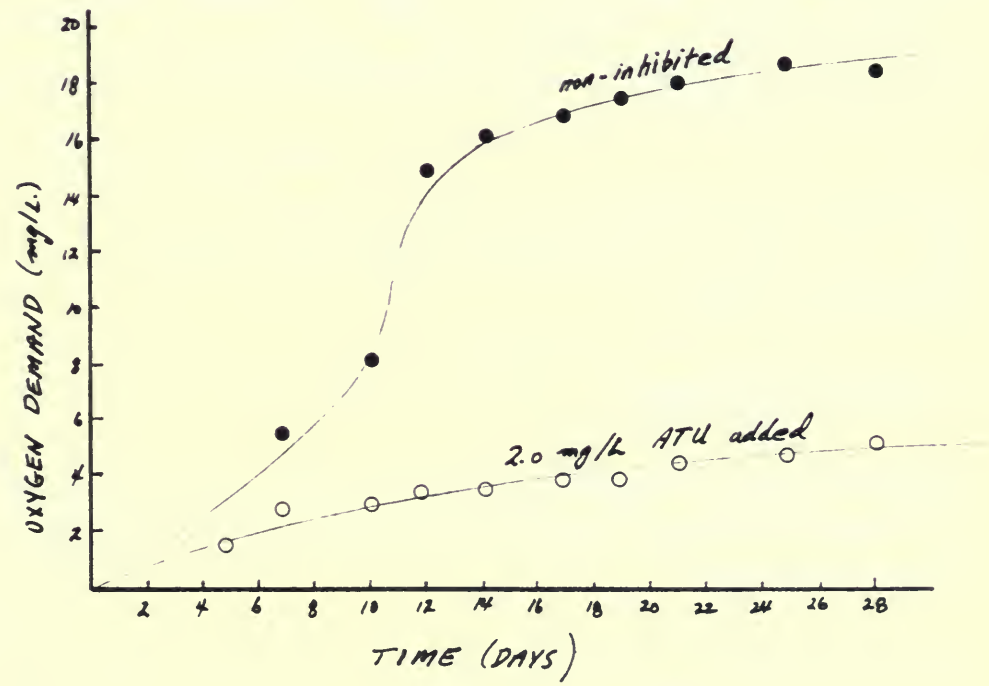


Figure 4-

BOD WITH NITRIFICATION INHIBITION



The reliability of the subsequent data interpretation will depend on the suitability of the technique applied in the laboratory analysis described above. For the determination of $CBOD_u$ the data required from the laboratory analysis is the carbonaceous BOD exerted (cumulative oxygen demand) vs. time. In the first example above, (Figure 4.6), the data are taken from the first-stage curve, and its extrapolated values. In the second example (Figure 4-7), the ATU-inhibited curve is used.

There are about 22 different methods available for evaluating the long-term BOD data. Most of these make the assumption that the first-stage BOD curve may be interpreted as a first order reaction of the form:

$$y_t = L(1 - 10^{-k_1 t}) \qquad (21)$$

in which

y_t = the oxygen demand exerted in time t (= $CBOD_t$);
L = the ultimate BOD of first-stage demand (= $CBOD_u$);
k_1 = the rate constant related to base 10.

The relationship between $CBOD_u$ and $CBOD_5$ is thus seen to be:

$$\frac{CBOD_u}{CBOD_5} = \frac{L}{y_5} = \left(1 - 10^{-5k_1}\right)^{-1} \qquad (22)$$

Two methods[1] will be described here for the evaluation of k_1 and L: the Thomas slope method and the Thomas Graphical linearization method.

A. Thomas Slope Method

The long-term BOD (carbonaceous) exertion curve data are tabulated as in table 4-2 below, using the explanations given on the page following the table.

---

1 Both the methods described are taken from Gaudy et al (1).

TABLE 4-2

Calculations of k_1 and L
(Thomas Slope Method)

Time (1)	Δ t (2)	Y (3)	Δ Y (4)	Y' (5)	YY' (6)	Y^2 (7)
0.79	0.79	0.20	0.20	0.46	0.092	0.04
1.29	0.50	0.5	0.30	0.70	0.35	0.25
1.79	0.50	0.9	0.40	2.10	1.89	0.81
2.29	0.50	2.6	1.70	3.90	10.12	6.76
2.79	0.50	4.8	2.20	2.60	12.48	23.04
3.29	0.50	5.2	0.40	0.84	4.37	27.04
3.96	0.67	5.8	0.60	0.743	4.25	33.64
5.00	1.04	6.3	0.50	0.481	3.03	39.69
6.08	1.08	6.8	0.50	0.318	2.16	46.24
8.08	2.00	6.9	0.10	0.064	0.442	47.61
11.20	3.12	7.2	0.27	0.048	0.34	51.41
14.20	3.00	7.2	0.03			
Totals:			47.17	12.243	39.524	276.53

N = number of time steps = 11

EXPLANATION
(Table 4-2)

Column No.

1 time, days;

2 Δt = difference in time between successive time measurements;

3 Y = observed BOD at stated time;

4 ΔY = difference in BOD between successive observations;

(n = order of values appearing in the table)

5 $$Y' = \frac{(Y_n - Y_{n-1})\left(\frac{t_{n+1} - t_n}{t_n - t_{n-1}}\right) + (Y_{n+1} - Y_n)\left(\frac{t_n - t_{n-1}}{t_{n+1} - t_n}\right)}{t_{n+1} - t_{n-1}}$$

$$Y' = \frac{(\Delta Y)_{n-1}\left(\frac{\Delta t_{n-1}}{\Delta t_{n+1}}\right) + (\Delta Y)_{n+1}\left(\frac{\Delta t_{n+1}}{\Delta t_{n-1}}\right)}{\Delta t_{n-1} + \Delta t_{n+1}}$$

From Table 4-2:

$\Delta Y_{n-1} = 0.20$
$\Delta Y_{n+1} = 0.30$
$\Delta t_{n+1} = 0.50$
$\Delta t_{n-1} = 0.79$

e.g., for first value of Y' in the table

$$Y' = \frac{(0.2)\frac{(0.79)}{(0.50)} + (0.3)\frac{(0.50)}{(0.79)}}{0.79 + 0.50} = .392$$

YY' = (COL. 3) x (COL. 5) = 39.524

Y^2 = (COL. 3)^2 = 276.53

The column totals $\Sigma \Delta Y'$, $\Sigma Y'$, $\Sigma YY'$ and ΣY^2 are substituted in the following two normal equations, which are then solved for a and b:

$$Na + b\Sigma\Delta Y - \Sigma Y' = 0 \qquad (23)$$

$$a\Sigma\Delta Y + b\Sigma Y^2 - \Sigma YY' = 0 \qquad (24)$$

substituting from the data:

$$11a + 47.17b - 12.343 = 0$$
$$47.17a + 276.53b - 39.524 = 0$$

Solving simultaneously:

$$a = 1.86$$
$$b = -0.175$$

The values of k_1 and L are then derived from the relationships:

$$L = \frac{a}{-b} \qquad (25)$$

$$k_1 = -.4343\ b \qquad (26)$$

By substituting the values of a and b in equations 25 and 26 we obtain:

$$k_1 = -.4343(-.175) = 0.76 \quad \text{per day, (to base e)}$$
$$L = 1.86 \div -(-.175) = 10.63 \quad \text{mg/L}$$

The computed value of k_1 can be used in equation 22 to compute the ratio of $CBOD_u/CBOD_5$ as follows:

$$\frac{CBOD_4}{CBOD_5} = \left(1 - 10^{-k_1(5)}\right)^{-1} = \left(1 - 10^{-0.76(5)}\right)^{-1}$$

$$= \underline{\underline{1.71}}$$

B. Thomas Graphical Method

In the example below, the same data are used as for the example above. Table 4-3 shows the required data tabulation. A linear plot is made of $(t/Y)^{1/3}$ vs time and the slope and the intercept are determined (figure 4-8). The values of L and k_1 are readily determined as:

$$k_1 = 2.61\ b/a \quad \textbf{(per day to base e)} \tag{27}$$

$$L = 1/(2.3\ k_1 a^3) \tag{28}$$

where a is the intercept and b is the slope, determined from the graph. From the example plot, a = .73 and b = .0369; therefore:

$$k_1 = 2.61\left(\frac{0.0369}{0.73}\right) = \underline{0.132}\ \textbf{(per day to base e)}$$

$$L = 1/\left[2.3(0.132)(0.73)^3\right]$$

$$= \underline{8.47}\ \text{mg/L}$$

Hence, for this method, the $(CBOD_u/CBOD_5)$ ratio

$$= (1 - 10^{-5k_1})^{-1}$$

$$= (1 - 10^{-5x0.132})^{-1}$$

$$= \underline{1.28}$$

Table 4-3

Calculations of k_1 and L
(Thomas Graphical Method)

Time (t)	Y	$\frac{t}{Y}$	$(t/Y)^{1/3}$
(1)	(2)	(3)	(4)
0.79	0.2	3.95	1.58
1.29	0.5	2.58	1.385
1.79	0.9	1.99	1.257
2.29	2.6	0.881	0.961
2.79	4.8	0.581	0.834
3.29	5.2	0.633	0.858
3.96	5.8	0.683	0.881
5.00	6.3	0.794	0.928
6.08	6.8	0.894	0.964
8.08	6.9	1.171	1.054
11.20	7.17	1.562	1.160
14.20	7.2	1.972	1.254

Column	Remarks
1	time in days;
2	observed BOD at stated time;
3	Col. 1/Col. 2;
4	$(Col.3)^{1/3}$.

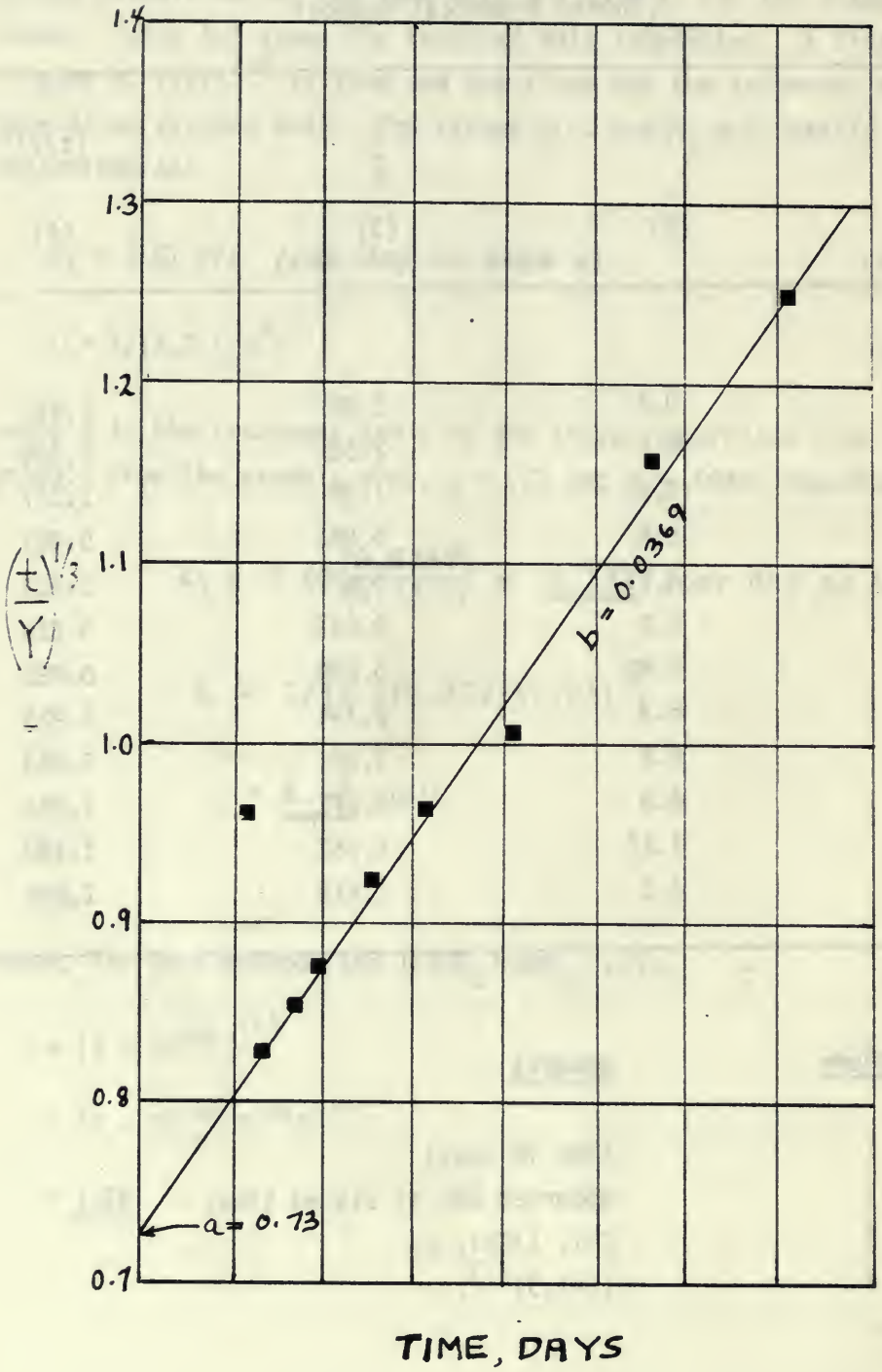


Figure 4-8

Determination of Constants a and b
for [illegible] raphical Method

4.4.3 Required Model Parameter: L_o

Having derived the $CBOD_u/CBOD_5$ ratio, the required model parameter L_o can be defined. L_o designates the ultimate carbonaceous oxygen demand at the head of a reach. This is calculated as a mass balance of the $CBOD_u$ of the water upstream of the point source, and of the point source effluent itself.

$$\text{i.e.:}\quad L_o = \frac{Q_u C_u + Q_i C_i}{Q_u + Q_i} \tag{29}$$

where Q_u = flow rate from upstream;
Q_i = point source discharge rate;
C_u = $CBOD_u$ of upstream water;
C_i = $CBOD_u$ of point source effluents.

The $CBOD_u$ of the point source effluent and upstream water is best determined from a long-term BOD of samples. The BOD_u is a long and difficult test to perform and therefore it is advisable to limit the number of samples submitted. It is best to take the effluent sample first, and a sample at the downstream end of the study reach second. Data from the latter test can be used to estimate the $CBOD_u/CBOD_5$ ratio, and the ratio applied to an upstream $CBOD_5$ to estimate $CBOD_u$ (C_u in equation 29).

Example

A mass balance is required to estimate the theoretical L_o below a WPCP outfall.

Known:

k_1	= .22 day^{-1} (base e)
Flow from WPCP	= 27.0 L/sec
Streamflow above WPCP	= 347.0 L/sec
BOD_5 of WPCP effluent	= 19.5 mg/L
Stream BOD_5 above WPCP	= 1.35 mg/L

$$BOD_5 \text{ (mixed)} = \frac{(1.35)(347.0) + (27.0)(19.5)}{347.0 + 27.0} = 2.66 \text{ mg/L}$$

$$BOD_u \text{ (mixed)} = 2.66 \times (1 - e^{-k_1 t})^{-1}$$

$$= 2.66 \times (1 - e^{-.22(5)})^{-1}$$

$$= 3.99 \text{ mg/L}$$

4.4.4 Required Model Parameter: N_o

The ultimate biochemical oxygen demand of nitrogenous substances is much easier to determine than it is for carbonaceous matter. The complete oxidation to NO_3 of one gram of nitrogen in an organic form is a two step process requiring 4.57 grams of dissolved oxygen. In the first step the ammonia group, NH_3, is oxidized to nitrite, NO_2, requiring 3.43 grams O_2 per gram of N, and in the second step, the NO_2 is further oxidized to NO_3, requiring 1.14 grams O_2 per gram N. (See also section 1.8).

To derive the ultimate NOD at the head of the study reach, N_o, a mass balance is made, using the above stoichiometric relationships and sampling data from the point source and from upstream of the point source.

$$n_1 = \left[\frac{Q_u (NO_2)_u + Q_i (NO_2)_i}{Q_u + Q_i}\right] \times 1.14 \qquad (30)$$

where

n_1 = nitrite-N oxygen demand;
Q_u = upstream flow rate;
Q_i = point source discharge rate;
$(NO_2)_u$ = upstream nitrite-N concentration;
$(NO_2)_i$ = point source effluent nitrite-N concentration.

$$n_2 = \left[\frac{Q_u K_u + Q_i K_i}{Q_u + Q_i}\right] \times \ldots \qquad (31)$$

where

n_2 = Kjeldahl-N oxygen demand;
K_u = upstream TKN concentration;
K_i = point source effluent TKN concentration;

Therefore: $N_o = n_1 + n_2$ (32)

Since nitrite occurs in only small quantities in a majority of situations, it is usually acceptable to calculate N_o based on TKN alone.

Example:

Estimate the theoretical N_o below a WPCP outfall (assuming insignificant nitrite -N concentrations)

Known:	TKN of WPCP effluent	=	24.5 mg/L
	Stream TKN above WPCP	=	0.62 mg/L
	Flow from WPCP	=	27.0 L/sec
	Streamflow above WPCP	=	347.0 L/sec

$$\text{TKN (mixed)} = \frac{(27.0)(24.5) + (347.0)(.62)}{347.0 + 27.0} = 2.34 \text{ mg/L}$$

N_o (mixed) = 2.34 x 4.57
= 10.69 mg/L

4.5. ESTIMATION OF RATES

4.5.1 Required Model Parameters: K_r and K_d

The rate coefficient K_r represents the overall rate of CBOD removal from the water column; it will always be greater than or equal to K_d, the rate of CBOD removal solely due to de-oxygenation. To establish these rates, BOD_5 data and time of travel data from the study reaches are analysed in the following manner.

The BOD_5 remaining (in kg./day) at each station is plotted on semi log paper vs. time-of-travel (see fig. 4-9a). An additive effect on the BOD_5 load will be evident at the time-of-travel where other sources occur. The subsequent calculations are done on a reach-by-reach basis, the reaches delineated by these additional inputs, if present. For each reach, plotted points are fit with a straight line. The slope of this line represents the K rate for the reach; therefore

$$K = \frac{\ln (BOD_{(T1)} / BOD_{(T2)})}{T_2 - T_1} \quad (day^{-1} \text{ to base e}) \qquad (33)$$

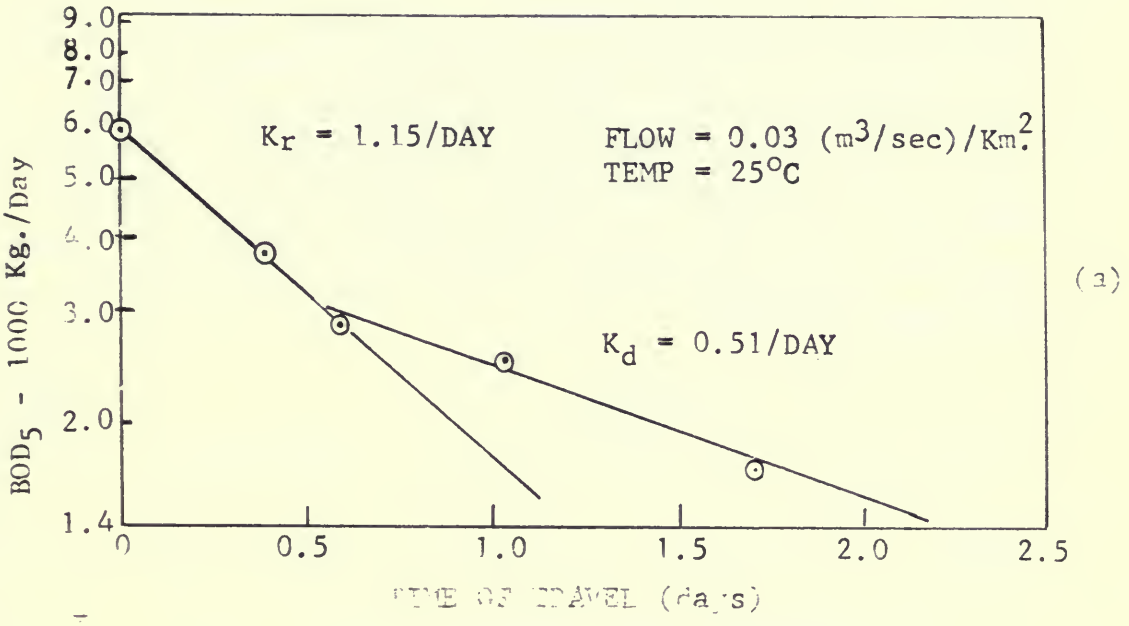
where T_1 and T_2 represent time of travel at start and end of reach and $BOD_{5(T1)}$ and $BOD_{5(T2)}$ are the means of survey results of BOD_5 at start and end of reach, respectively.

If the first reach K calculated is much higher than those following and has a value greater than 1.0 day^{-1}, then this is evidence of some process in addition to de-oxygenation, acting on the stream BOD. This higher loss rate is generally the result of volatile suspended solids (VSS) settling to the stream bottom, or less commonly, by adsorption to plants. Settling out is a potentially important process when effluent contains a high concentration of settleable waste solids, as with pulp mill wastes. Verification of BOD loss by settling can be made by plotting VSS data as for BOD_5 data (see figure 4-9b). If significant losses of suspended solids coincide with a high loss rate of BOD, then settling is an important factor. In this case, K_d is less than K_r, and K_d is evaluated based on the slope of the next reach downstream.

In the example, therefore, K_r = 1.15 per day for the first 0.6 day travel time, and K_d = 0.51 per day for the entire 2.5 day travel time. K_r is assumed equal to K_d after the first 0.6 day travel time.

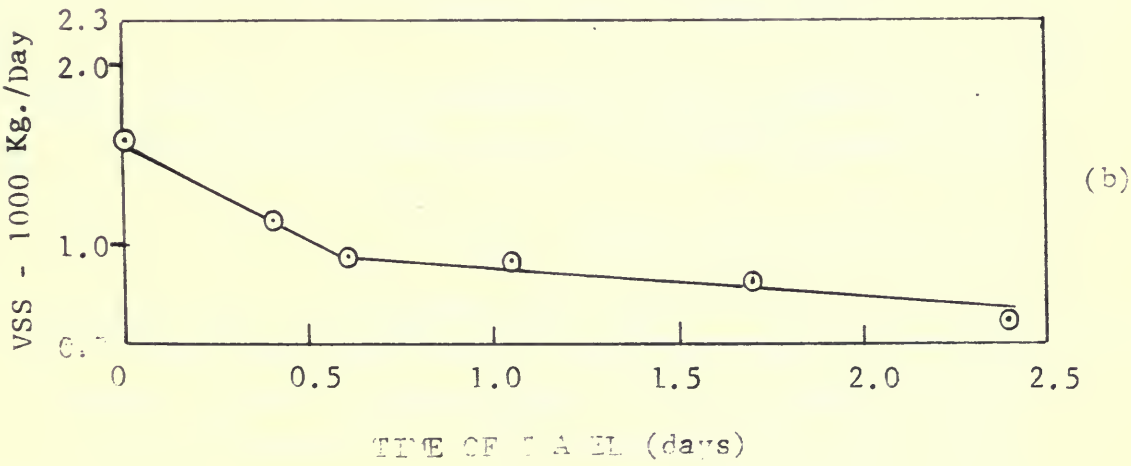
Figure 4-9

(a) Log Plot of BOD vs Time of Travel



(a)

(b) Log Plot of VSS vs Time of Travel



(b)

4.5.2 Required Model Parameter: S

As stated in the previous section, benthic deposits will be found where K_r is greater than K_d. In addition to being found immediately below sources of effluent high in settleable solids, benthic deposits are also found in impounded areas, and at river bends. The oxygen demand rate per unit of area is usually determined by direct measurements in situ with the benthic respirometer at strategic locations in the study section. The actual rate applied is the product of this measured rate and the ratio of the area covered by benthal deposits to the total area of the reach. To convert from this areal rate to a volumetric rate as used in the steady-state model, the areal rate is divided by the mean depth as follows:

$$S = \frac{\bar{S} \cdot r}{H} \qquad (34)$$

where

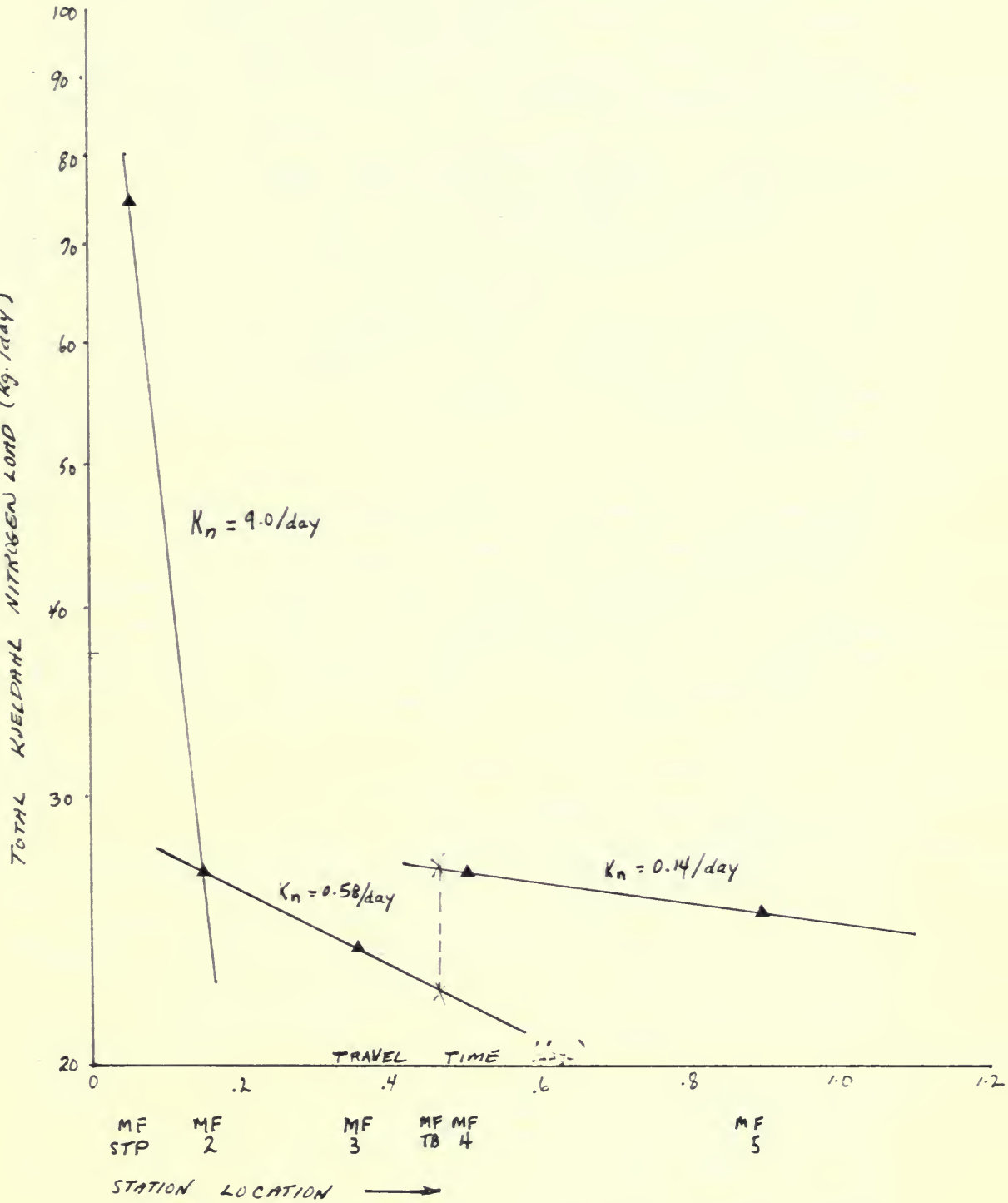
S = sediment oxygen demand rate (mg/L per day);
$\bar{S}$ = measured sediment oxygen demand rate (g/m^2 per day);
r = ratio of area of benthal deposits to total area;
H = mean depth in the reach (m).

The proportion, r, is also sometimes determined by relating the mean width of benthal deposits in a reach to the mean reach width

4.5.3 Required Model Parameter: K_n

The coefficient K_n represents the instream rate of oxygen demand resulting from the oxidation of reduced nitrogen forms (Nitrification). There are various methods by which it may be estimated including long term BOD analysis similar to the technique used in ultimate CBOD determination. A more reliable method, however, is to directly apply the field data by constructing a semi-log plot of TKN transport (kg/day) vs. time of travel over the study zone (see figure 4-10).

(per day to base e)



This plot generally results in a series of straight line segments making evident reaches over which nitrification is more or less uniform. The K_n rate for each reach thus distinguished is calculated by selecting points from the corresponding line segment and calculating the slope as:

$$K_n = \frac{\ln (TKN_1 / TKN_2)}{T_2 - T_1} \quad (day^{-1} \text{ base } e) \qquad (35)$$

where T_1 and T_2 represent the time of travel at the start and end of the reach and TKN_1 and TKN_2 are the means of survey results of Kjeldahl-N at the start and end of the reach, respectively.

The rates will be found to vary considerably, and depend on the physical characteristics and hydraulic aspects of the stream sections and upon the prevalence of other nitrogen transformations which may be occurring. Consequently, some knowledge of the instream processes affecting nitrogen is necessary. These processes are discussed in section 1.8 of the manual.

4.5.4 Required Model Parameter: K_a

There are numerous methods for obtaining a direct measurement[1] of the reaeration coefficient, K_a, in the field. However, since K_a is one of those parameters which may be varied in order to calibrate the steady-state model, additional costly field work may be by-passed by applying more approximate estimation techniques.

Many empirically derived relationships exist which give a reasonable estimate of K_a, provided a method is chosen which is suitable for the study river. The main consideration is whether or not the hydraulic characteristics of the river are similar to those of the rivers from which data were used to derive the empirical equation.

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1 See section 3.2.6 for one method: the ethylene-dye tracer technique.

Figure 4-11

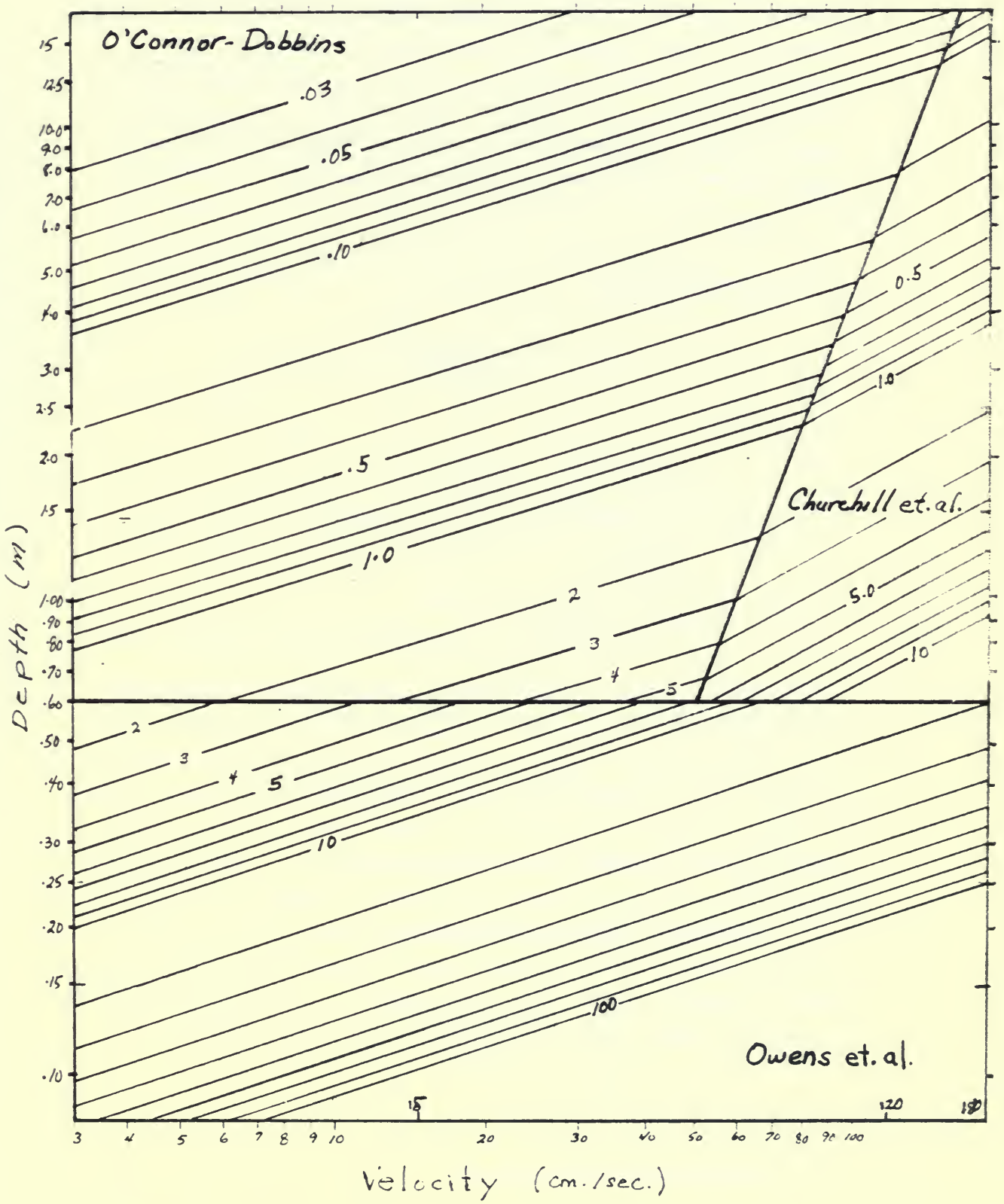

O'Connor-Dobbins
.03
.05
.10
.5
1.0
2
3
4
5
10
100
0.5
1.0
5.0
10
Churchill et.al.
Owens et.al.
Depth (m)
Velocity (cm./sec.)

Figure 4-11 is a key[1] to selecting an appropriate method from three recommended empirical formulae, based on stream velocity and average depth. The equation of Owens et al is used for streams (less than approximately 0.6 m in depth), while for streams with greater depths, either the O'Connor-Dobbins, or the Churchill et al equation is used, depending on which area of the figure best describes the hydraulic properties of the stream in question. The value of K_a may be read from the figure directly, or calculated by applying the corresponding formula, as given below. Note that all equations, and the figure, give K_a to the base e.

(i) O'Connor-Dobbins

$$K_a = \frac{1.34\ (D_m U)^{0.5}}{H^{1.5}} \qquad (36)$$

where

D_m = the molecular diffusion coefficient of O_2 in water (cm^2/hr)

U = stream velocity (cm/sec)

H = mean stream depth (m)

The molecular diffusion parameter D_m is temperature dependent, calculated from:

$$D_m = 8.7 \times 10^{-2} \exp\left[0.159\ (T_c - 20.0)\right]\ cm^2/hr \qquad (37)$$

where T_c = water temperature in degrees C.

(ii) Churchill, Elmore and Buckingham

$$K_a = \frac{0.058\ U^{0.969}}{H^{1.673}} \qquad (38)$$

---

1 This figure taken from Covar (2).

(iii) Owens, Edwards and Gibbs

$$K_a = \frac{0.245\,U^{0.67}}{H^{1.85}} \qquad (39)$$

Temperature Effects

While the O'Connor-Dobbins calculation of K_a takes temperature into account in the expression for molecular diffusion, the other two formulae do not. When estimated by the Churchill et al or Owens et al equations, K_a is the reaeration coefficient at 20^oC. To correct K_a for other temperatures the following equation is used:

$$K_{a\,(T)} = K_{a\,(20^oC)}\ 1.024^{(T-20)} \qquad (40)$$

where:

T = observed temperature;
$K_{a(T)}$ = K_a at observed temperature;
$K_{a(20)}$ = K_a at 20^oC.

4.5.5 Required Model Parameters: t_s and p:

The times of sunrise (t_s) and sunset for any locality may be determined by reference to tables in "The Observers Handbook" published by the Royal Astronomical Society. Knowing the latitude of the locality, the tables are entered to obtain sunrise and sunset time in Local Mean Time (LMT). The LMT is corrected to Local Standard Time by obtaining a correction, in minutes of time from the Standard Time table also contained in the Handbook. The correction is added or subtracted from the LMT depending on whether the locality is west or east, respectively, of the standard meridian. The corrections for any place not listed may be obtained by finding its longitude from a map and converting to time (360^o = 24 hour).

The fraction of the day which is daylight, p, is simply the time difference between sunrise and sunset divided by 24 hours. If a pyranograph (sunshine intensity recording instrument) is available, the times of sunrise, sunset and daylight period, may be obtained directly from the charts (see Figure 4-12).

Example:

Find the time of sunrise (t_s), sunset, and p for Owen Sound, Ontario for June 11.

Referring to the table of standard times in the Handbook the following entry is found for Owen Sound:

	LAT.	CORR.
OWEN SOUND	45^{o}	+ 24E

which indicates that Owen Sound is in the Eastern time zone (E), and requires a correction of 24 minutes. Referring to the tables of sunrise and sunset times for 45^{o} latitude, we find that sunrise is at 0413 and sunset at 1947. Therefore for Owen Sound sunrise is at 0437 and sunset is at 2011, adding the 24 minute correction.

The proportion of the day for sunlight is:

$$p = \frac{\text{20 hrs. 11 min. - 4 hrs. 37 min.}}{\text{24 hrs.}} = .65$$

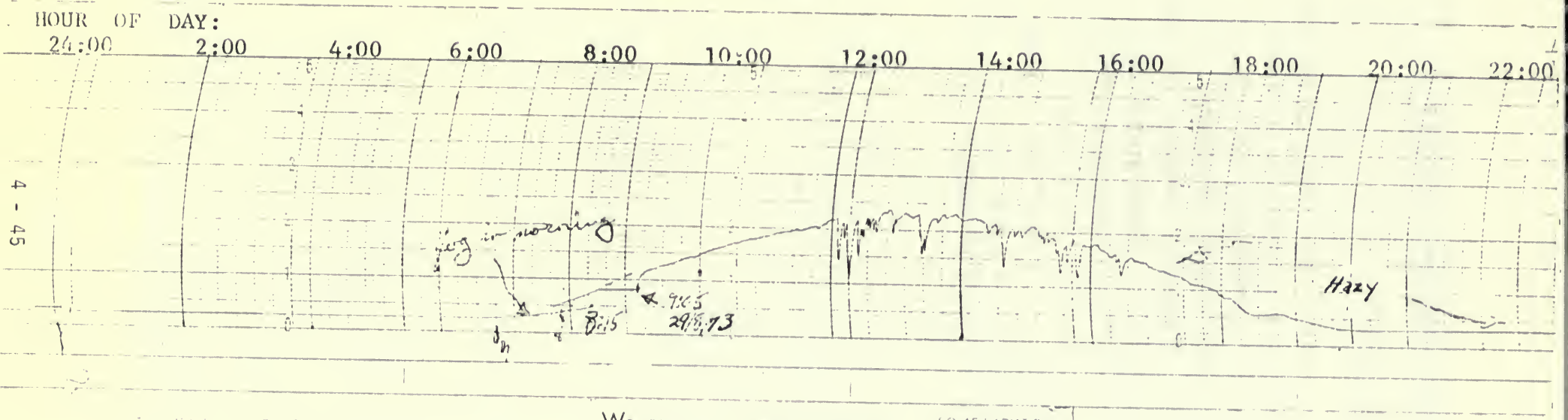
Note that if the first table had contained no entry for Owen Sound we could have calculated the correction knowing the west longitude, which is 81^{o}. The Standard Meridians occur every 15^{o} west of Greenwich, and for the Eastern zone, 75^{o} is the Standard Meridian. Therefore the correction is:

$$\frac{6^{o}}{360^{o}} \times 24(hr) \times 60\left(\frac{min}{hr.}\right) = 24\ min.$$

and the sign is + since Owen Sound is west of 75^{o} west longitude.

PYRANOGRAPH: SUNLIGHT INTENSITY

STATION: Mount Forest DATE: 29/8/73



WEATHER MEASURE CORPORATION
P O BOX 41[illegible] SACRAMENT CALIFORNIA 95841
TELEPHONE 9[illegible] 481 7565

SOLAR RADIATION
RECORDER CHART
#C401-W

STATION: Singleton Mt. Forest Met DATE ON 29/8/73 DATE OFF

4 - 45

Mar 80

4.5.6 Required Model Parameters: P_{av} and R

A. Diurnal Curve Analysis

The necessity for estimating the rates of photosynthetic oxygen production and respiration by plants can be determined from a preliminary analysis of continuous DO monitoring data. Significant plant effects are indicated by diurnal fluctuations in DO greater than about 2.0 mg/L. The model parameter P_{av} is defined as the average rate of photosynthetic oxygen production in mg/L per day resulting from the activity of submerged aquatic plant growth. The respiration rate, R, is defined as the average rate of DO uptake by plants in mg/L per day, and is the difference between gross community respiration and all oxygen sinks other than plant respiration. Both parameters are estimated by the analysis described below.

The method given here for estimating rates of photosynthesis and respiration is the single station method of diurnal curve analysis. To perform the analysis, the model parameters K_a, and t_s must first be determined, as described in foregoing sections. Having obtained these, a continuous DO and temperature record of about 48 hours, from the downstream station of the study reach is selected for analysis and plotted (see Figure 4-13). Data should be selected which includes normally sunny days, such that the DO curve is free from anomalies.

The plotted data is next used to produce a second plot of the rate of change of oxygen concentration versus time, using the approximation:

$$\frac{dc}{dt} = \frac{c_2 - c_1}{t_2 - t_1} \qquad (41)$$

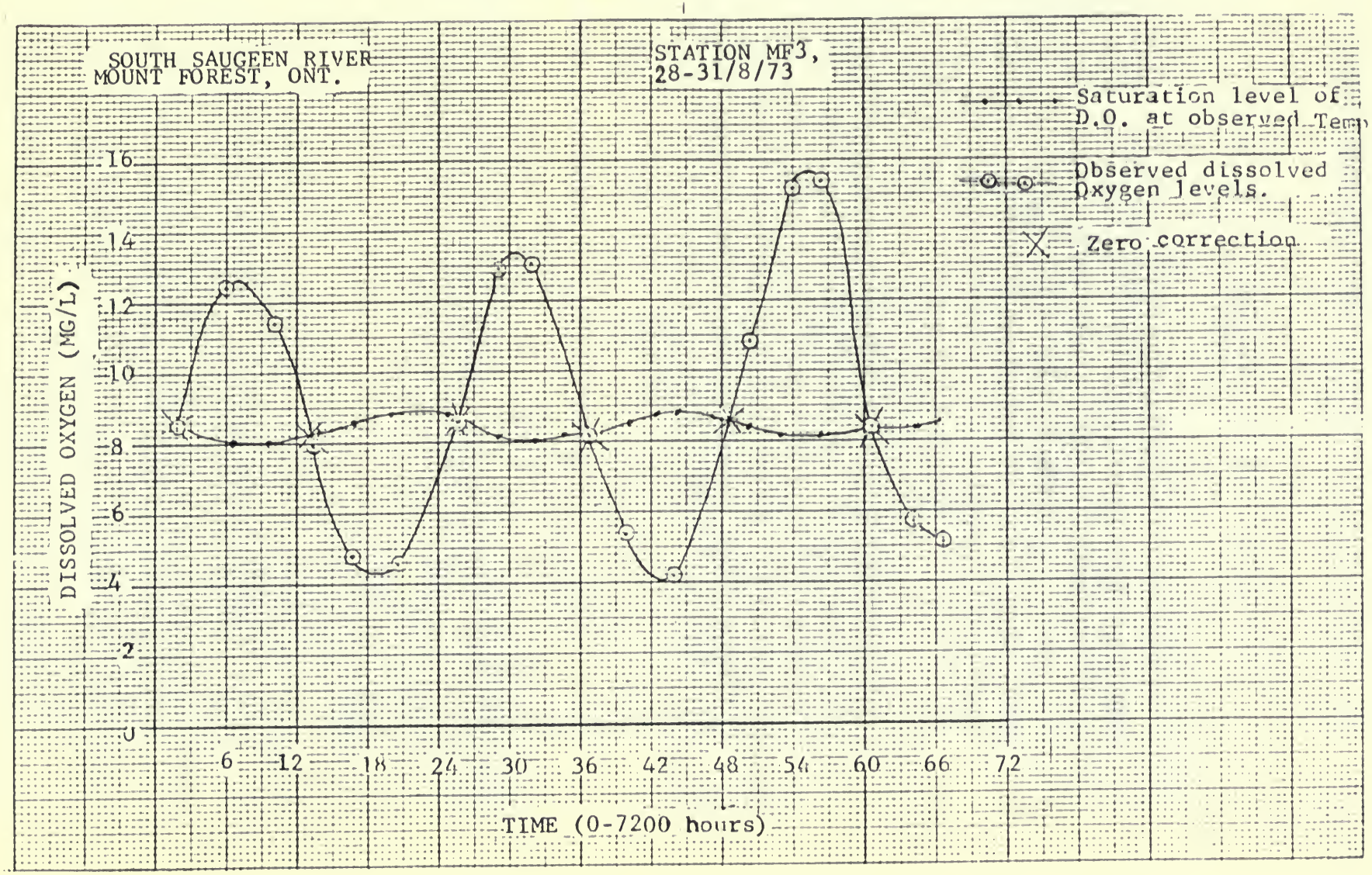
where:

dc/dt = rate of change of DO concentration;

c_1, c_2 = oxygen concentration in mg/L at times t_1 and t_2, respectively.

Figure 4 - 13

PLOT OF DISSOLVED OXYGEN AND SATURATION
DISSOLVED OXYGEN CONCENTRATION



The rate of change for each pair of data is plotted versus the mean survey time of that pair of data; i.e.,

$$\text{plotting time (in hours)} = \frac{t_1 + t_2}{2} \qquad (42)$$

Extra points should be plotted near sunrise and sunset, as dc/dt changes rapidly at these times. Figure 4-14 shows the dc/dt vs. time curve (uncorrected) for the data of Figure 4-13. The curve should be plotted on squared paper with reference to a zero rate of change axis, as dc/dt can be positive or negative. The value of dc/dt is positive when DO is increasing and negative when DO is decreasing (i.e., positive when $c_2 > c_1$).

A smooth curve may be drawn through the DO data (as in Figure 4-13) to facilitate this analysis. At the maximum and minimum DO indicated by the curve, the rate of change, dc/dt, is zero. These points are generally plotted first at exactly their times of occurrence. The plot should be extensive enough to clearly indicate two successive night-time minima.

The next step in the procedure is to correct the dc/dt curve for gaseous diffusion. The rate of diffusion is equal to the reaeration rate (K_a) times the oxygen deficit. Therefore,

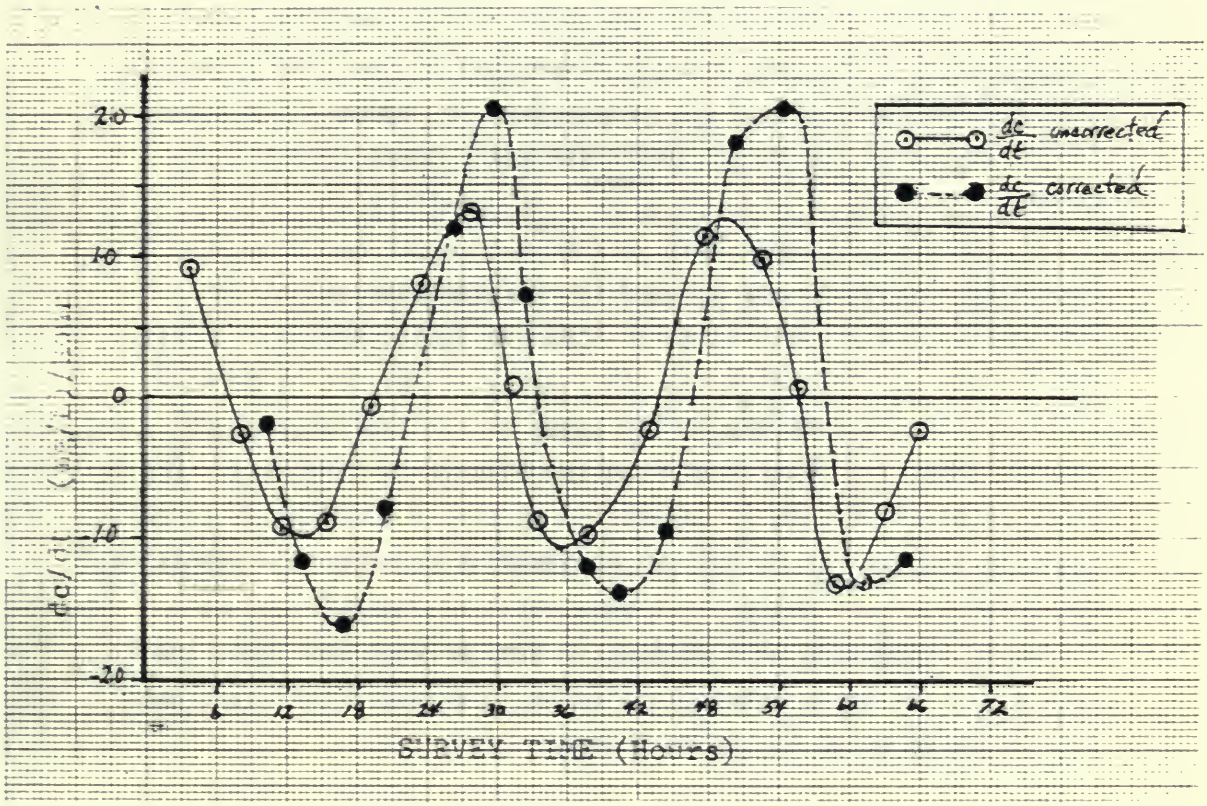
$$\frac{dc}{dt_c} = \frac{dc}{dt} - K_a \left(c_s(t) - c(t) \right) \qquad (43)$$

where

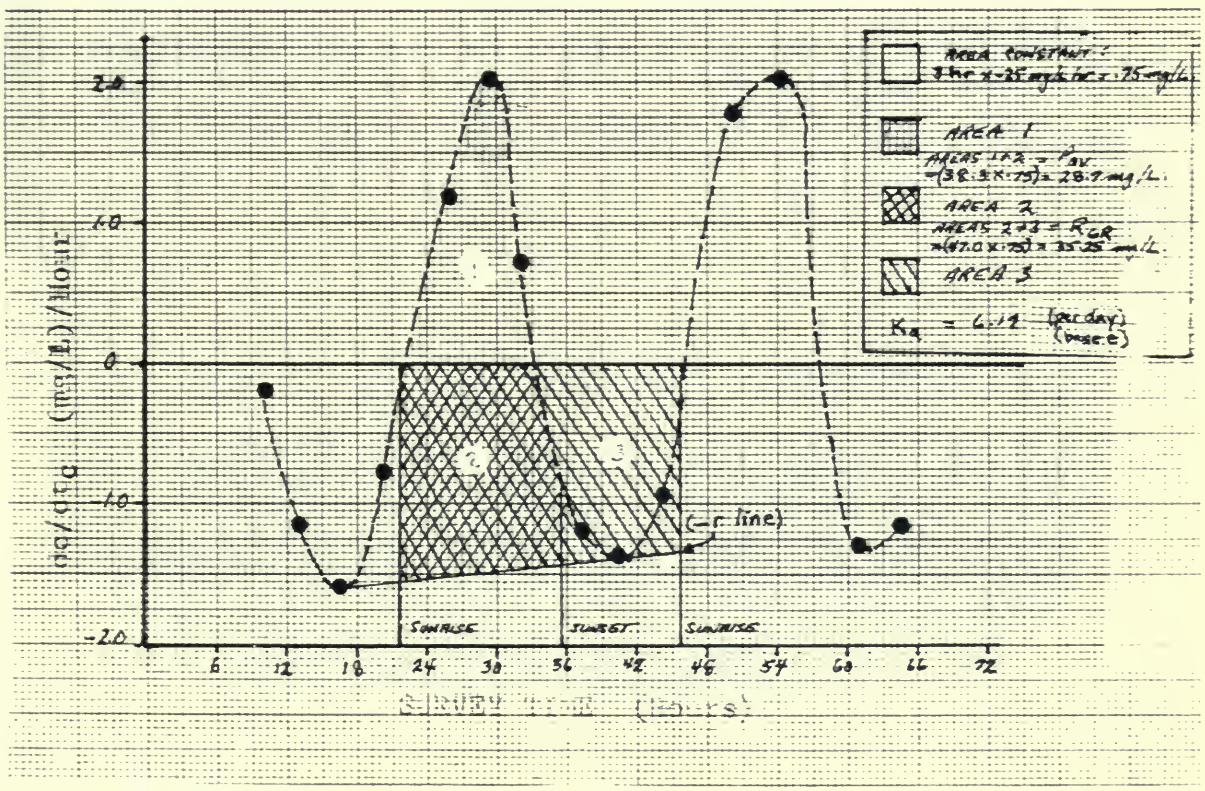
dc/dt_c	=	the corrected rate of change DO;
K_a	=	coefficient of reaeration (per hour);
$c_s(t)$	=	DO saturation concentration at time t (for observed water temperature);
$c(t)$	=	observed DO concentration at time t.

Figure 4 - 14
EXAMPLE PLOT ANALYSIS
STREETER-PHELPS SURVEY DATA (A. / P.)

(a) Correction of dc/dt curve for reaeration



(b) Integration of dc/dt_c curve



In Figure 4-13, a smooth curve has been drawn through the theoretical c_s values. At the points where the saturation curve intersects the DO curve (ie. $c_s(t) = c(t)$) the correction is 0. The plotting times chosen for the dc/dt_c curve should include these 0 correction times and the mid-points of the time intervals used for the dc/dt plot.[1] At these times, equation 43 is solved for dc/dt_c using the DO concentration c(t), taken from the DO curve.

The dc/dt_c curve can now be used to calculate the amount of respiration and production which occurred during the day. First, the 24-hour day for which the rates apply is defined by marking vertical lines across the dc/dt_c curve at the two consecutive sunrise times (as determined from the Observer's Handbook). The gross respiration rate is determined by simply assuming it to be equal to the average of the two consecutive pre-dawn low points in the dc/dt_c curve. Hence, if a straight line joining the two minima is integrated with respect to time between sunrises, the result will be gross respiration in mg/l per day. This straight line is referred to as the -r line.

Daily gross photosynthesis for the plotted day is calculated by integrating between the -r line and the dc/dt_c curve beginning at sunrise. In practice, the above integrations are done by counting the squares in the appropriate areas of the curve and multiplying by the square constant. Figure 4-14 illustrates how the dc/dt_c curve has been divided into areas representing gross photosynthesis and community respiration.

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1 A program suitable for use with a table-top programmable calculator is available for calculating points on the curve. A program which performs the entire diurnal curve analysis is presently being developed. Personnel doing extensive P & R calculations from single station data should contact the Regional Services Unit for these programs.

B. Calculation of P_{av}, P_{max} and R

(a) P_{av}: The gross photosynthesis estimated by the above technique is the average daily rate.

(b) P_{max}: Although P_{av} is adequate for input to the DOMOD3 model, it is actually the maximum rate of photosynthesis (P_{max}) which is used in the algorithm. P_{max} is calculated from P_{av} based on an assumption of sine function variation in photosynthetic rate during the day, and constrained by the average rate:

$$P_{max} = P_{av}\left(\frac{\pi}{2P}\right) \tag{44}$$

(c) R: The respiration rate calculated on the basis of the above single station data analysis will include the effects of all oxygen sinks in the reach. To calculate net respiration for the purposes of the steady state model, the effects of sediment oxygen demand, nitrogenous and carbonaceous oxygen demand must be deducted from the gross respiration rate. The following equations are used for this calculation.

(i) If $K_r = K_d$;

$$R_{NET} = R_{GR} - S - \left[\frac{L_o(1-e^{-K_d T}) + N_o(1-e^{-K_n T})}{T}\right] \tag{45}$$

where

R_{GR} = Gross respiration (from diurnal curve analysis) (mg/L per day);
L_o = Ultimate CBOD at upstream station (mg/L);
N_o = Ultimate NOD at upstream station (mg/L);
K_d = Coefficient of CBOD removal per day (to base e);
K_n = Coefficient of NOD removal per day (to base e);
S = Benthic respiration (mg/L per day);
T = Time of travel between stations (day).

(ii) If $K_r > K_d$;

$$R_{NET} = R_{GR} - S - \left[\frac{\frac{K_d}{K_r} \cdot L_o \left(1 - e^{-K_r T}\right) + N_o \left(1 - e^{-K_n T}\right)}{T}\right] \quad (46)$$

Example:

Calculate net respiration for a reach with the following measured characteristics.

Known:
R_{GR} = 14 mg/L.day
L_o = 8.0 mg/L
S = 5.0 mg/L.day
N_o = 5.6 mg/L
K_r = 1.15 per day (base e)
K_d = .5 per day (base e)
K_n = .39 per day (base e)
T = .5 day

Using equation 46:

$$R_{NET} = 14 - 5.0 - \left\{\frac{\frac{5}{1.15}\left[8.0\left(1 - e^{-1.15(0.5)}\right)\right] + 5.6\left(1 - e^{-.39(0.5)}\right)}{0.5}\right\}$$

= 6.64 mg/L per day

Note that: the equations used above consist of the same terms used in the steady-state DO-BOD model, modified to exclude the effects of reaeration, since the graphical analysis uses the rate of change data corrected for reaeration.

4.6 MODEL CALIBRATION AND VERIFICATION

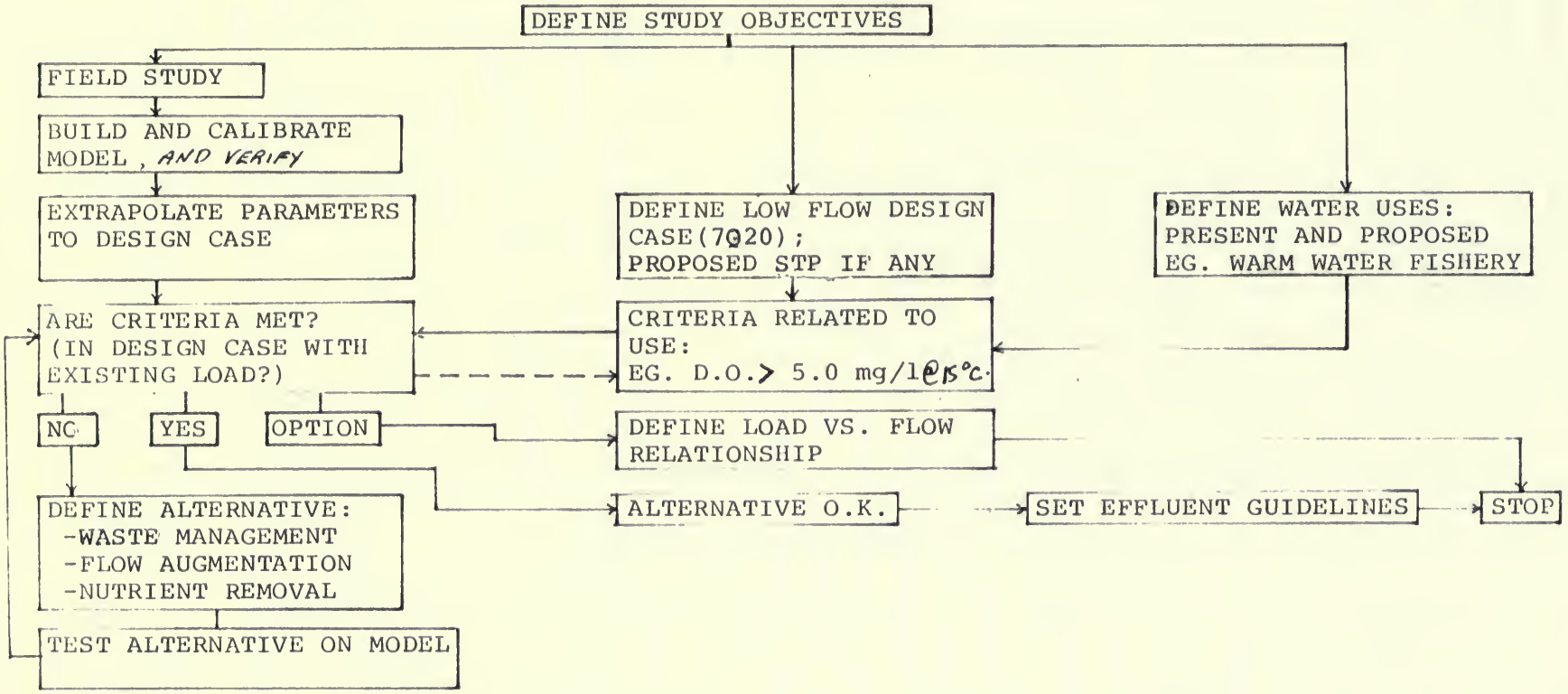
4.6.1 Model Calibration

The general pattern followed in developing and applying the steady-state DO simulation model to a particular case is illustrated in figure 4-15. To be applied to its ultimate objective of determining allowable waste loadings to achieve a specified

Figure 4-15

IDEALIZED USE OF STEADY STATE MODELLING TO

EVALUATE WATER QUALITY MANAGEMENT ALTERNATIVES


DEFINE STUDY OBJECTIVES
FIELD STUDY
BUILD AND CALIBRATE MODEL, AND VERIFY
EXTRAPOLATE PARAMETERS TO DESIGN CASE
DEFINE LOW FLOW DESIGN CASE(7Q20); PROPOSED STP IF ANY
DEFINE WATER USES: PRESENT AND PROPOSED EG. WARM WATER FISHERY
ARE CRITERIA MET? (IN DESIGN CASE WITH EXISTING LOAD?)
CRITERIA RELATED TO USE: EG. D.O. > 5.0 mg/l @ 15°C
NO
YES
OPTION
DEFINE LOAD VS. FLOW RELATIONSHIP
DEFINE ALTERNATIVE:
-WASTE MANAGEMENT
-FLOW AUGMENTATION
-NUTRIENT REMOVAL
ALTERNATIVE O.K.
SET EFFLUENT GUIDELINES
STOP
TEST ALTERNATIVE ON MODEL

receiving water quality, the model must first be calibrated in such a way that it can reliably simulate observed stream DO under a variety of conditions. Calibration entails making reasonable adjustments to certain mathematical functions within the model in such a way as to achieve a good fit of modelled results to observed data; following this, verification is attempted using an independent set of data, wherever possible.

The data derived by the methods described in previous sections of the manual can be categorized as being (a) entirely reliable (b) reasonably accurate, or (c) rough approximation. Some physical parameters or variables, such as temperature, stream flow rate and waste loading rate are the controlling variables for the DO simulation model, and as such may be thought of as indisputable (category (a)). Other data, such as observed DO, is reasonably accurate and may be assumed correct within the limits of measurement error (category (b)). Finally, the data usually most subject to error are the process rates, which in addition to measurement error, have a high degree of spatial and temporal variability; in addition, error may be injected into their calculation by the use of unavoidably imperfect estimation techniques, or by laboratory error (category (c)).

Twelve parameters and rates, plus upstream DO and saturation DO concentrations (c_s) are input to the DOMOD3 model; of these, photosynthesis (average), respiration (net) and reaeration are the three rates usually adjusted to obtain model calibration. Where plant activity is minimal, the task of calibration reduces to adjusting K_a. In studies with no P and R terms, minor adjustments to K_r and K_d may also be permitted, particularly if K_a has been measured in the field by a reliable method. Where plant activity is significant and for stream reaches where there is much shading by trees, the time of sunrise, t_s, and fraction of day between sunrise and sunset, p, may also be changed for these reaches.

First attempts at obtaining a fit of model output to observed DO's rarely succeed. However, a careful examination of the differences will often reveal the steps necessary to obtain a good fit. An input data sheet similar to that shown in table 4-4 should be prepared and updated for each model run during the calibration process.

Table 4 - 4

INPUT DATA SHEET FOR DOMOD3 RUN

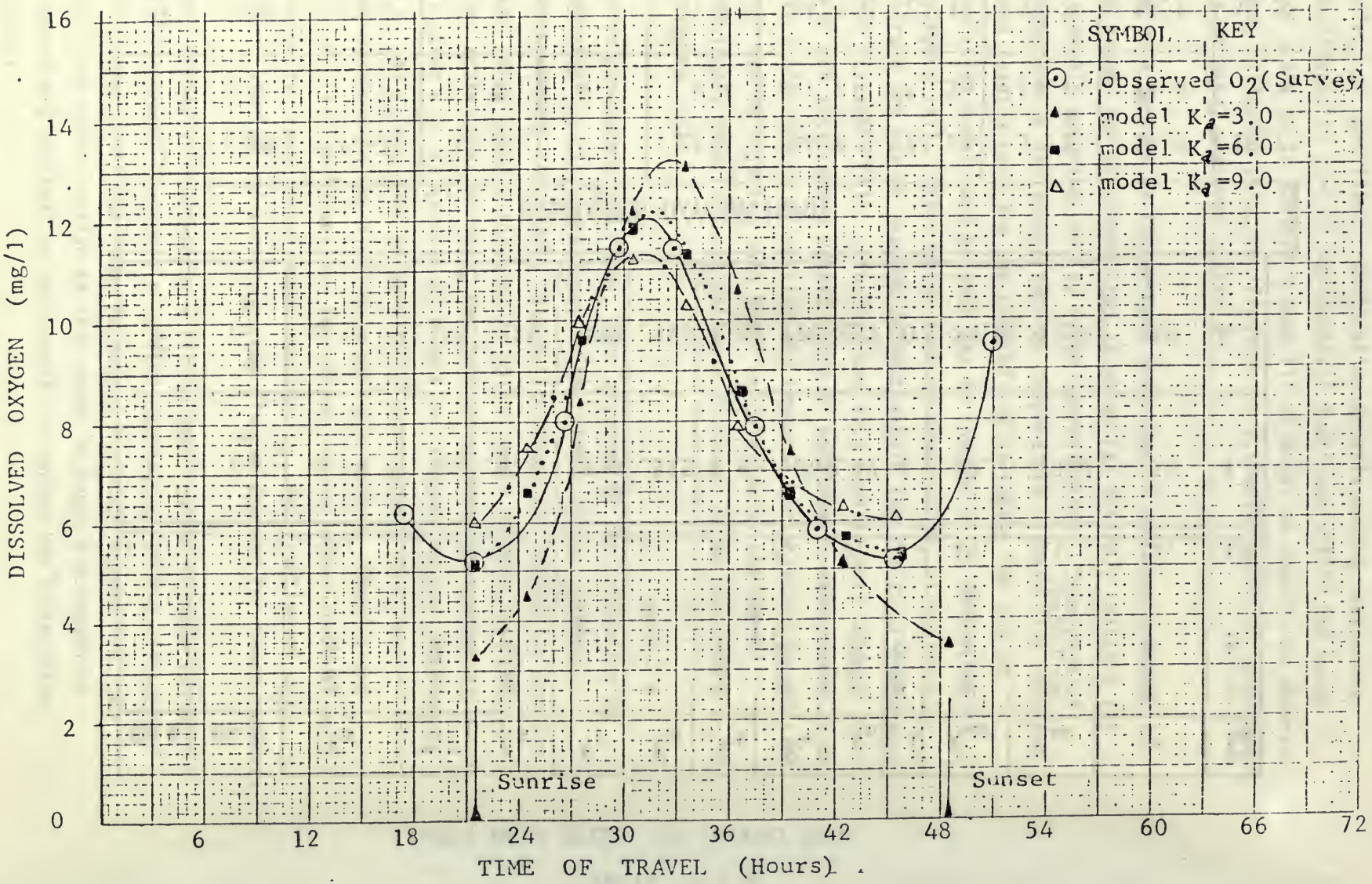
Run #	Sta. #	C_s	T_oT	t_s	K_r	K_d	L_o	K_n	N_o	S	R_{Nor}	ρ_{av}	ρ	K_a
		mg/L	days	days	1/day	e 1/day	mg/L	e/day	mg/L	mg/L	mg/L/day	mg/L/day	Days	e/day
		1.	2.	3.	4.	5.	6.	7.	8.	9.	10.	11.	12.	13.
1	5 to 6	8.33	.388	.271	.525	.525	1.8	.20	2.42	0	20.0	20.0	.58	3.0
2	5 to 6	8.33	.388	.271	.525	.525	1.8	.20	2.42	0	20.0	20.0	.58	6.0

DISSOLVED OXYGEN INPUT

1	TIME	12.2	15.2	18.2	21.2	24.2	27.2	30.2	33.2	36.2
	D.O.	7.8	5.2	4.5	4.9	7.2	11.3	13.1	12.0	8.5
2	TIME	12.2	15.2	18.2	21.2	24.2	27.2	30.2	33.2	36.2
	D.O.	7.8	5.2	4.5	4.9	7.2	11.3	13.1	12.0	8.5

Figure 4 - 16

EFFECT OF VARYING REAERATION COEFFICIENT K_a
- CALIBRATION RUN OF DO MODEL



K_a should be varied first. Increasing K_a will cause the output DO curve to flatten and peak earlier; decreasing K_a causes the opposite effect (see figure 4-16). Care should be exercised if it becomes necessary to vary P and R. Photosynthesis should always exceed net respiration to be realistic, and adjustment of these rates should be roughly within the confidence limits of the techniques used to estimate the rates. Figure 4-17 illustrates examples of the effects on output of changing various model parameters. Figure 17(a) shows the output DO from a model run using a particular parameter set. In each subsequent run, (b) through (f), only one or two parameters are varied from the original parameter set. Increases in P will be reflected in increased steepness of the DO peak (see Figure 4-17 (f)), while changes in respiration, R, will shift the whole DO curve up or down (see Figure 4-17 (b)). Changing the plant respiration rate will result in the same effect as changing the benthic respiration rate, S, since both processes have identical terms in the mathematical formulation of the model (see Figure 4-17 (b) and (c)).

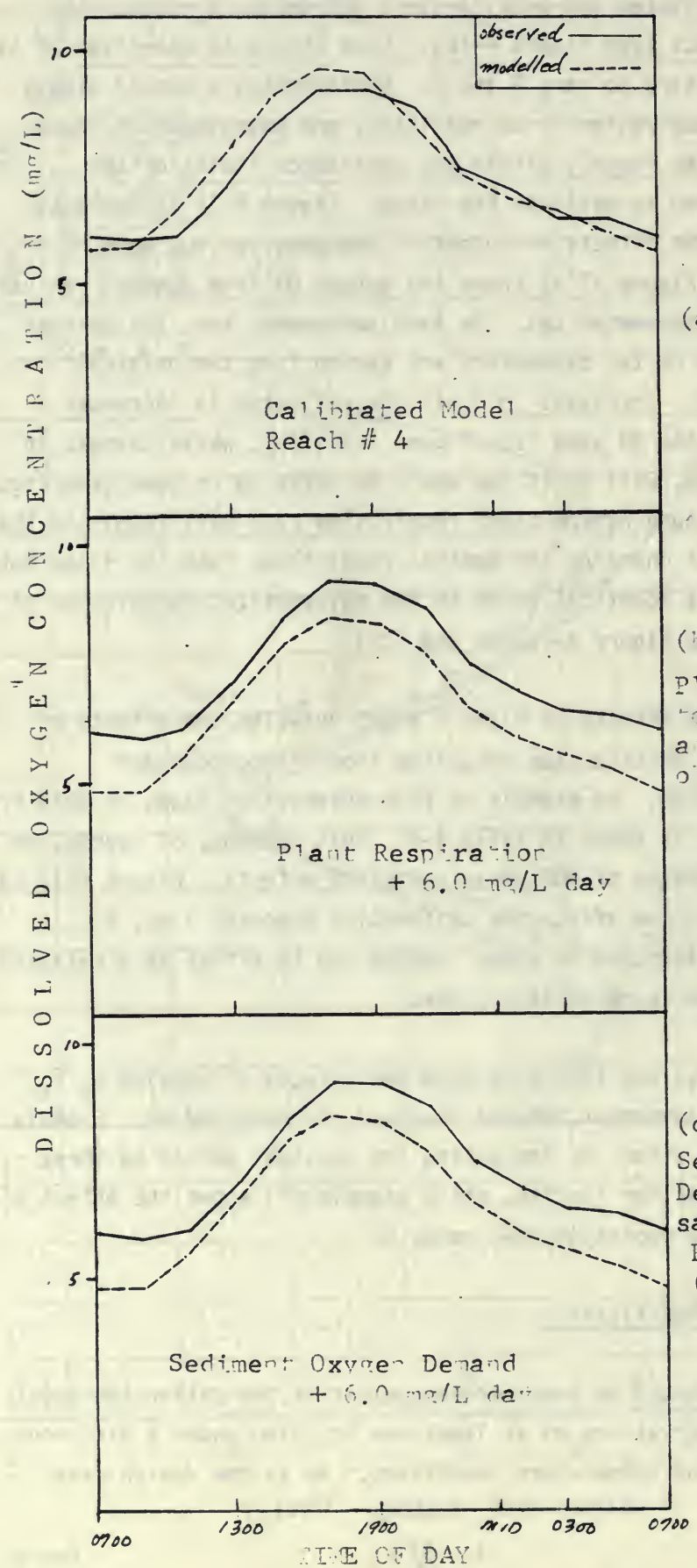
Sometimes, the effects of P and R might outweigh the effects of oxygen demand satisfaction resulting from nitrogenous and carbonaceous BOD. An example of this observation based on data from the Thames R. is shown in table 4-5. This depends, of course, on the relative rates of BOD decay and plant effects. Figure 4-17 (d) shows how this can affect the calibration process; i.e., a considerable increase in waste loading can be offset by a relatively small decrease in respiration rate.

Figure 4-17 (e) and (f), also show the effects of varying P, t_s, and p on the agreement between observed and modelled DO. Example (e) shows the effect of increasing the daylight period by three hours by an earlier sunrise, while example (f) shows the effect of increasing the photosynthetic rate, P.

4.6.2 Model Verification

Comparisons should be made between output of the calibrated model and field observations on at least one occasion under a different set of flow and temperature conditions. As in the design case

Figure 4 - 17
EFFECTS OF PARAMETER VARIATION
ON MODEL OUTPUT



(b) Plant Respiration too high ; poor agreement with observed DO's

(c) Sediment Oxygen Demand change has same effect as Plant R change in (b)

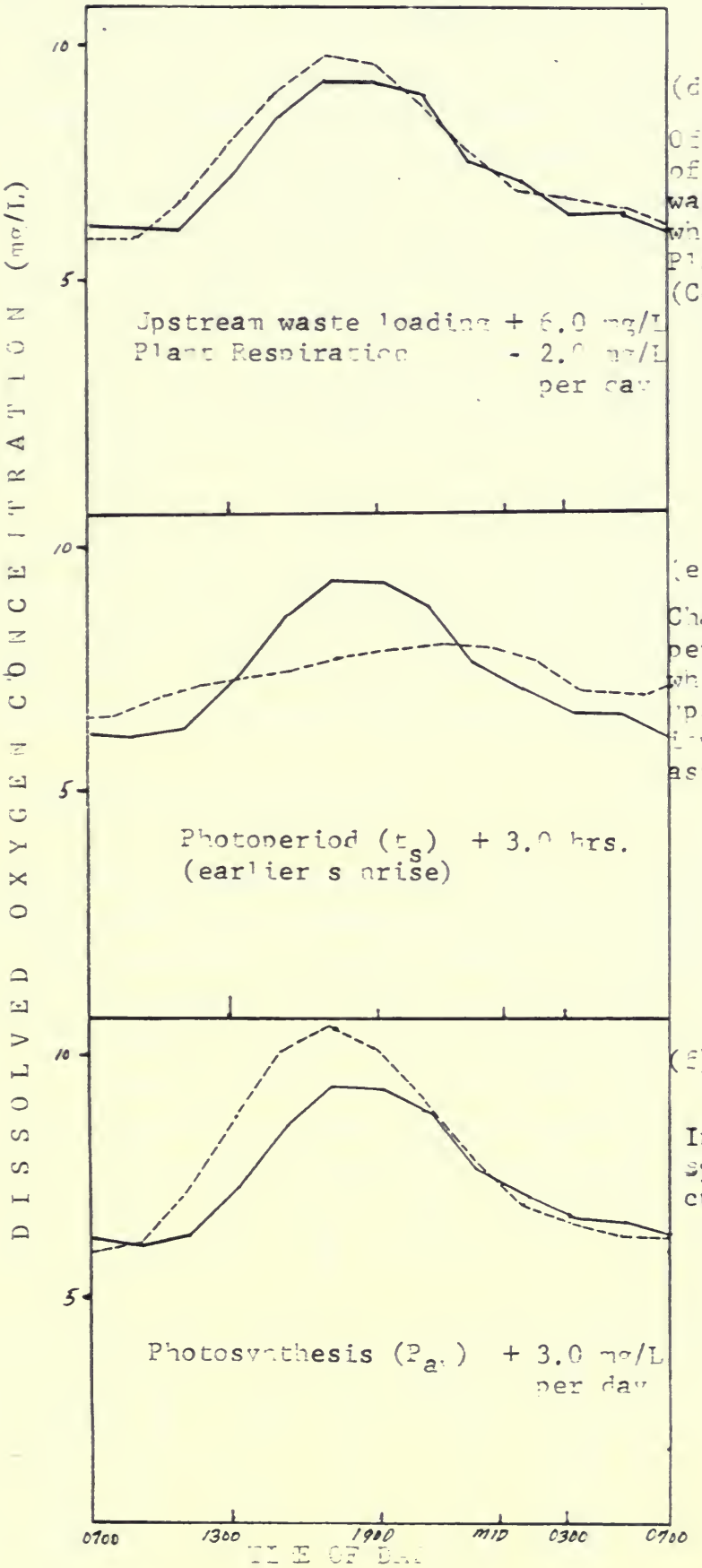

DISSOLVED OXYGEN CONCENTRATION (mg/L)
10
5
Upstream waste loading + 6.0 mg/L
Plant Respiration - 2.0 mg/L
per day
(d)
Offset the effects of increasing waste load (L_o) while lowering Plant respiration. (Compare with (a))
Photoperiod (t_s) + 3.0 hrs.
(earlier sunrise)
(e)
Changing photo-period parameters while maintaining upstream boundary limit DO's causes asynchronous
Photosynthesis (P_a) + 3.0 mg/L
per day
(f)
Increasing photo-synthetic rate causes curve to steepen
0700
1300
1900
MID
0300
0700
TIME OF DAY

Table 4-5

Estimated Rates of Oxygen Depletion Due to Benthic Respiration and Oxygen Demand Satisfaction at Selected Locations in the Thames River Basin

Location	Date of Field Survey	Minimum Dissolved Oxygen Measured at end of Reach (mg/l)	Respiration Rate (R)* (mg/l/day)	Oxygen Demand** Satisfaction Rate (OD) (mg/l/day)	Ratio of Rates R/OD
Avon River downstream from Stratford	June 1972	1.2	30	2.3	12.8
North Thames River downstream from St. Marys	June 1971	2.5	55	.9	61
Thames River downstream from Woodstock	August 1971	4.7	13	25.4	.5
Thames River (S. Branch) in London	June 1970	4.6	11.4	12.5	.9
North Thames in London	June 1970	5.2	8.	9.8	.8
Thames River downstream from London	June 1970	5.4	17.	2.9	5.9
Thames River in Chatham	Sept. 1971	5.9	14	.5	28

* Respiration is from a combination of aquatic plants, attached and floating algae, and sludge dwelling organisms.(S&R in the model formulation)

** Rate calculated from the sum of carbonaceous and nitrogenous oxygen demands at the upstream end of the reach in question (K_dL & K_nN.)

formulation of the model, described in section 4.8.1, reaction rates, travel time, daylight period and waste loading will all have to be re-estimated based on relationships previously established.

The model may be considered verified if it has successfully predicted the change in water quality after a significant change in waste inputs and/or in the physical environment of the stream has occurred. If the verification fails, further data may have to be collected, especially on process rates. The plotted relationships are then revised accordingly and the model accepted, provided the changes are justifiable. If these efforts also result in unsatisfactory verification, this model may be unsuitable for the situation at hand, requiring modifications to the model or use of a different model; these alternatives are beyond the scope of the present chapter.

If verification under several flow and temperature conditions can be obtained, the model may be said to be reliable. One of the goals of model development should be to produce models which are suitable for a wide variety of conditions. In this event, management decisions based on model predictions can be confidently made.

4.7 Modelling with Limited Data

It is generally preferable to conduct studies as previously described, involving an intensive survey and the collection of data for model verification. A greater degree of confidence may be placed in the outputs of the steady state model when it is supported by a comprehensive data base. Time and resource constraints may, however, necessitate short-cuts from the ideal plan. One alternative, where only the very minimum of field work is possible, is to use the "Graphical Procedure" as described in Appendix 2. This procedure is suited to situations in which the effects of aquatic plant growth are not significant.

Steady state modelling has occasionally been used in the absence of a complete data base. Provided approximations for the various rates etc. are made on an informed basis, this is an acceptable approach

when short-cutting is unavoidable. Rates might be extracted from a previous study of the same, or a similar stream or river situation. In some cases, it might be possible to initiate some data collection to verify a model developed with this approach. Where DO data from the study section is available, preliminary modelling based on approximated rates may identify key processes which may be essential to measure in the field.

4.8 MODEL APPLICATION

4.8.1 Parameter Estimation for Design Case

After calibration and verification of the model, it is applied to predicting stream DO response under a specified set of conditions. This set of conditions defines the "critical" or "design" case, so called because proposed treatment facilities are designed to provide acceptable water quality under these conditions. This section describes the methods used to estimate the model parameters and rates for the design case conditions.

Typically, design conditions involve high temperatures, low flows (e.g., average seven day low flow with a 95% probability of occurence, i.e. 7Q20), and waste loads which combine to produce the worst oxygen conditions (4). Since dilution ratio is one of the most important factors in the impact of pollutants on receiving water, it is usual to consider a low flow case. Guidelines for selecting the design flow criterion, and the statistical determination of drought flow are described separately in this manual as Appendix 1. Biological activity is more intense at higher temperatures, thus affecting most of the process rates. It is usual to consider a temperature of 23-25^{o}C for the design case. Waste load predictions are based on population projections, for WPCP's, or on expansion plans in the case of an industry.

Application of the model to the design case involves re-estimating parameters from their calibrated values for the design conditions. The following table lists the parameters and the recommended method used for their re-estimation.

4 - 62 Mar 80

TABLE 4-6

PARAMETER RE-ESTIMATION FOR DESIGN

Parameter/Rate	Method Recommended
P_{av}, S, R	Re-estimate mean depth; rate is a function of depth.
K_r	Assume equal to K_d if primary treatment facilities to be installed.
K_n, K_d	derived graphically based on observation that rates move upstream with increased time of travel
K_a	Re-estimate using depth, and velocity relationships, e.g. O'Connor - Dobbins.
L_o, N_o	Re-calculate using mass balance.
travel time (x/u) H, u	Extrapolate relationships derived from survey data (see section 4.3).
Upstream DO	historical data record
Temperature	historical data record; recalculate all rates and DO saturation

These methods only provide guidelines for re-estimation of parameters, and may be superceded when other information is available; e.g. if nitrification treatment is proposed, the design run should be made with an estimate of K_n lower than normally estimated, as experience has shown this to be valid.

Parameter: K_a

1. Estimate drought flow time of travel for each reach from log-log plots of time of travel versus flow (section 4.3).
2. Calculate mean reach velocity as reach length divided by time of travel.
3. Calculate mean reach depth (section 4.3).
4. Estimate K_a from reach mean depth and velocity using technique described in section 4.5.4.

Parameter: K_r

Since the implementation of treatment facilities generally involves at least primary settling, which removes 40-60% of settleable solids, K_r may be considered equal to K_d. This is equivalent to assuming that all oxygen-demanding materials are in the dissolved phase of the stream water. While the assumption may not be entirely valid, previous work (3) has shown modelled DO to be relatively insensitive to changes in this process rate coefficient.

Parameters: P_{av}, S, R

The following assumptions are inherent in the recommended method used to re-estimate P_{av}, R and S for the design case:

1. The areal rates of these biological processes do not change under the design conditions.
2. The volumetric rates of these processes is inversely proportional to mean depth.

Using photosynthesis as an example, the second assumption indicates $P_{av} \propto H^{-1}$ and as outlined in section 4.3.2 we have:

$$\frac{P_{av_2}}{P_{av_1}} = \frac{H_1}{H_2} \qquad (15)$$

Recalling the Leopold-Maddock relationship of depth to flow:

$$H = aQ^d \qquad (9)$$

and subsituting in equation 15:

$$P_{av_2} = P_{av_1} \left(\frac{Q_1}{Q_2}\right)^d \qquad (17)$$

This equation may also be used, substituting S or R for P_{av}, for the re-estimation of rates for S and R under the design conditions. The exponent, d, is derived by the method described in section 4.3.4.

Figure 4 - 1

DETERMINATION OF K1 RATES (per day to base e)

(SOUTH SAUGEEN RIVER STUDY DATA)

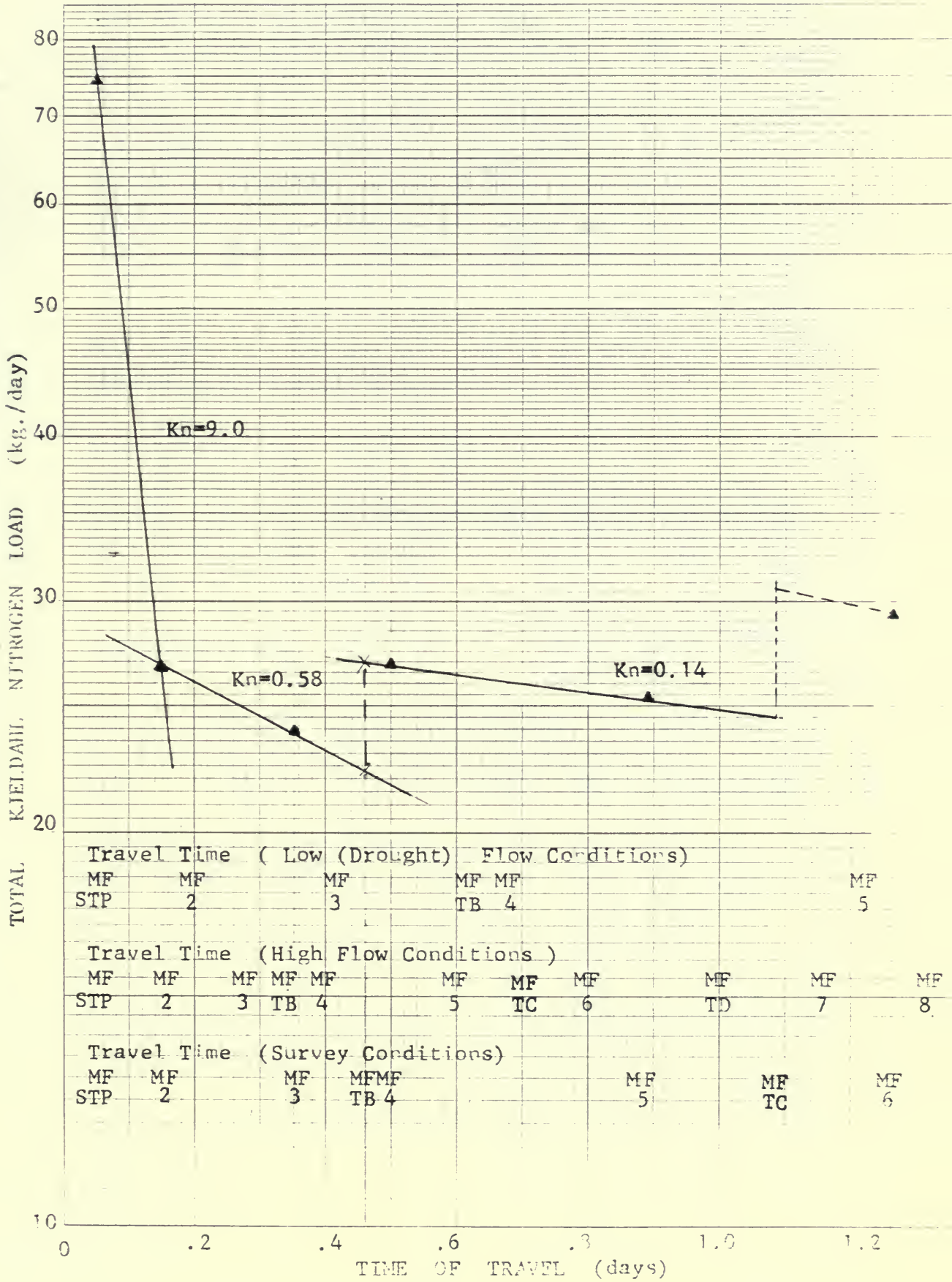
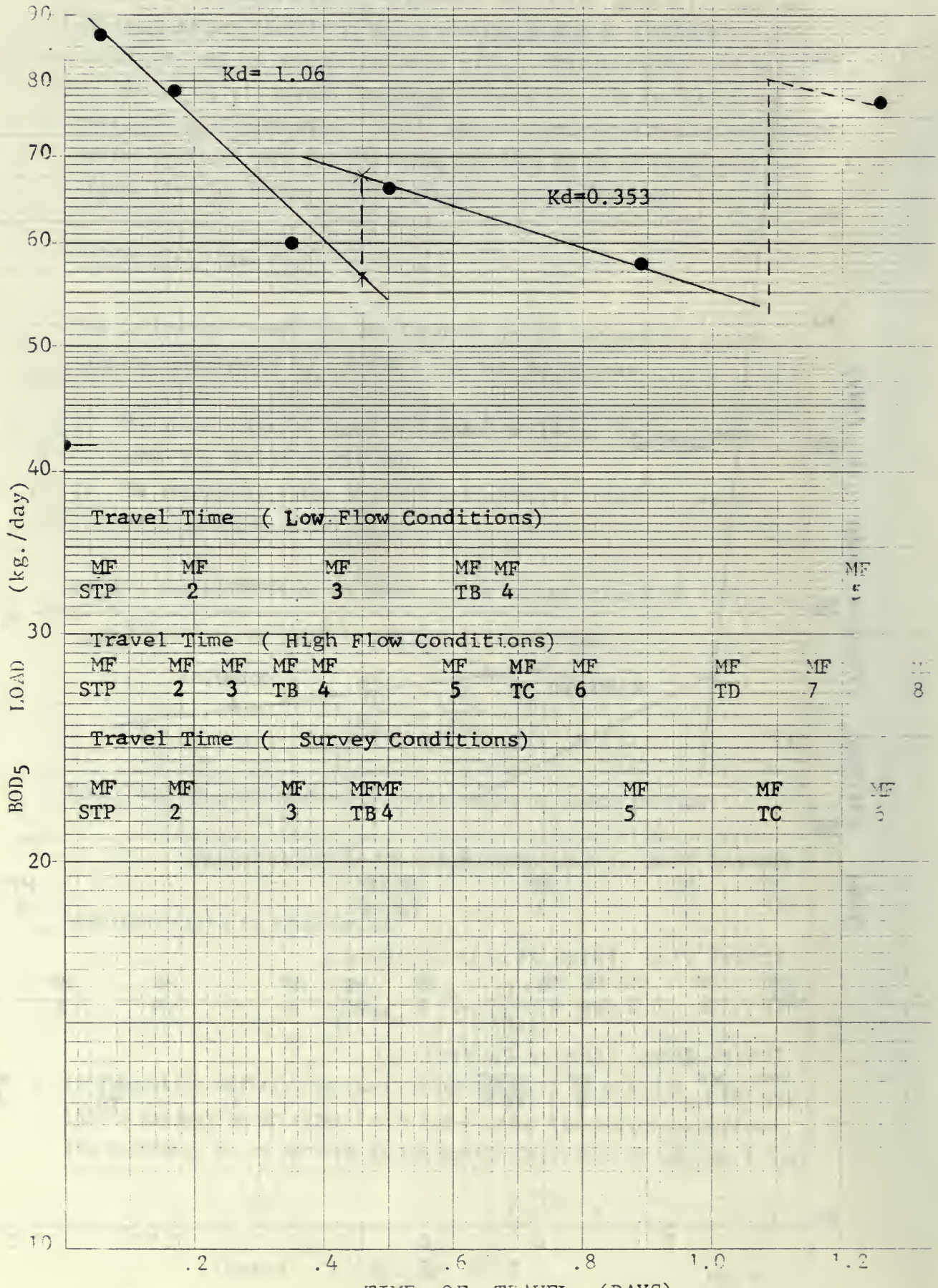


Figure 4 - 19

DETERMINATION OF K_d RATES (per day to base e)

(SOUTH SASKEEN RIVER SURVEY DATA)



Parameters: K_n and K_d

Under design conditions it is usually assumed that oxidation of NOD and CBOD occurs at the same rate as measured during survey conditions. Because of increased travel times at low flows however, other rates measured for a given reach at survey flow will apply to fraction of a complete reach further upstream. Practically, the best method to re-estimate the K_d and K_n rates is to plot CBOD and TKN on a log scale, versus time of travel, indicating station locations by time of travel on the abscissa. Different abscissa scales are drawn in for different flow conditions, as in figures 4-18 and 4-19; the appropriate K value can then be read directly. For example, in figure 4-18, at high flow conditions for the reach MFTB to MF4, K_n = 0.58 per day to base e; at low flow for the same reach, K_n = 0.14 per day to base e.

Note that these rates are also dependent on the hydraulic characteristics of the reach, but we are ignoring this in the approach described. For example, O'Connor suggests that the rate K_d is depth dependent. Correlations of K_n, K_d with Reynolds number are also published.

Temperature Effects

The water temperature during the design run must be estimated, based on drought flow temperature records for the study stream, or similar stream. The following processes are all affected by temperature: K_n, K_d, K_a, R and S. The rates generally will have been calculated from intensive survey data, and apply only at the prevailing water temperatures at the survey time. The exception to this is the reaeration coefficient which will apply at 20^o if calculated by the procedures described in section 4.5.4.

In general, the rate adjustment for design temperature is made using the following equation, based on van't Hoff's law:

$$K_{T2} = K_{T1} \cdot \theta^{T2 - T1} \qquad (47)$$

where:

K_{T2} = the kinetic rate at the design temperature;
K_{T1} = the kinetic rate at the survey temperature;
T_2 = the design temperature ^oC;
T_1 = the survey temperature ^oC;
θ = the temperature coefficient associated with the process (see table 4-7).

Several temperature coefficients for any one process are available from the literature, however the table below lists those values which have been selected and applied in past waste assimilation studies conducted by the Water Resources Branch.

4.8.2 Waste Loading Design

The desired end result of the work described in the previous chapters is the production of estimates of allowable waste loadings which will ensure acceptable water quality in the receiving water. Having derived estimates of all necessary reaction rates and other parameters for the design case, as described in the preceeding section, these loadings can be calculated. This is done by establishing upstream boundary (and tributary) DO conditions and running the model under the critical conditions and rates defined for each reach. The only variable in these runs is waste load, which is varied until the water quality of all the downstream reaches meets the required criteria.

TABLE 4-7

CHANGES IN RATES RESULTING FROM TEMPERATURE EFFECTS

Process Rate	Temperature Coefficient	Comments	Reference
R (net)	1.065		(4)
S	1.047		(4)
K_r	1.0 - 1.075	Increases as $K_r \rightarrow K_d$	(5)
K_d	1.075		(5)
K_a	1.024		(6)
K_n	1.097		(6)

The DO criteria specified by the MOE in the policy manual, "Water Management" (7) is given in table 4-8. What constitutes a violation of criteria for DO (or for certain other parameters) is a topic still under discussion. It could be argued that a DO "violation" is a function of magnitude (mg/L below a stated criterion) and duration. At present this is not explicitly defined in the criteria and the problem of interpretation must be handled on a case by case basis by the resource manager in consultation with biologists and policy analysts.

Figure 4-20 shows a useful presentation of the design case model results. This example provides a waste loading estimate for a fixed criterion of 4.0 mg/L under a range of stream flows and temperatures. Thus for any particular flow and temperature, a daily BOD loading is derived which, according to model predictions, will produce no occurrences of DO less than 4.0 mg/L.

Table 4-8

PROVINCIAL WATER QUALITY OBJECTIVES FOR DISSOLVED OXYGEN

At no time should dissolved oxygen concentrations be less than the values specified below:

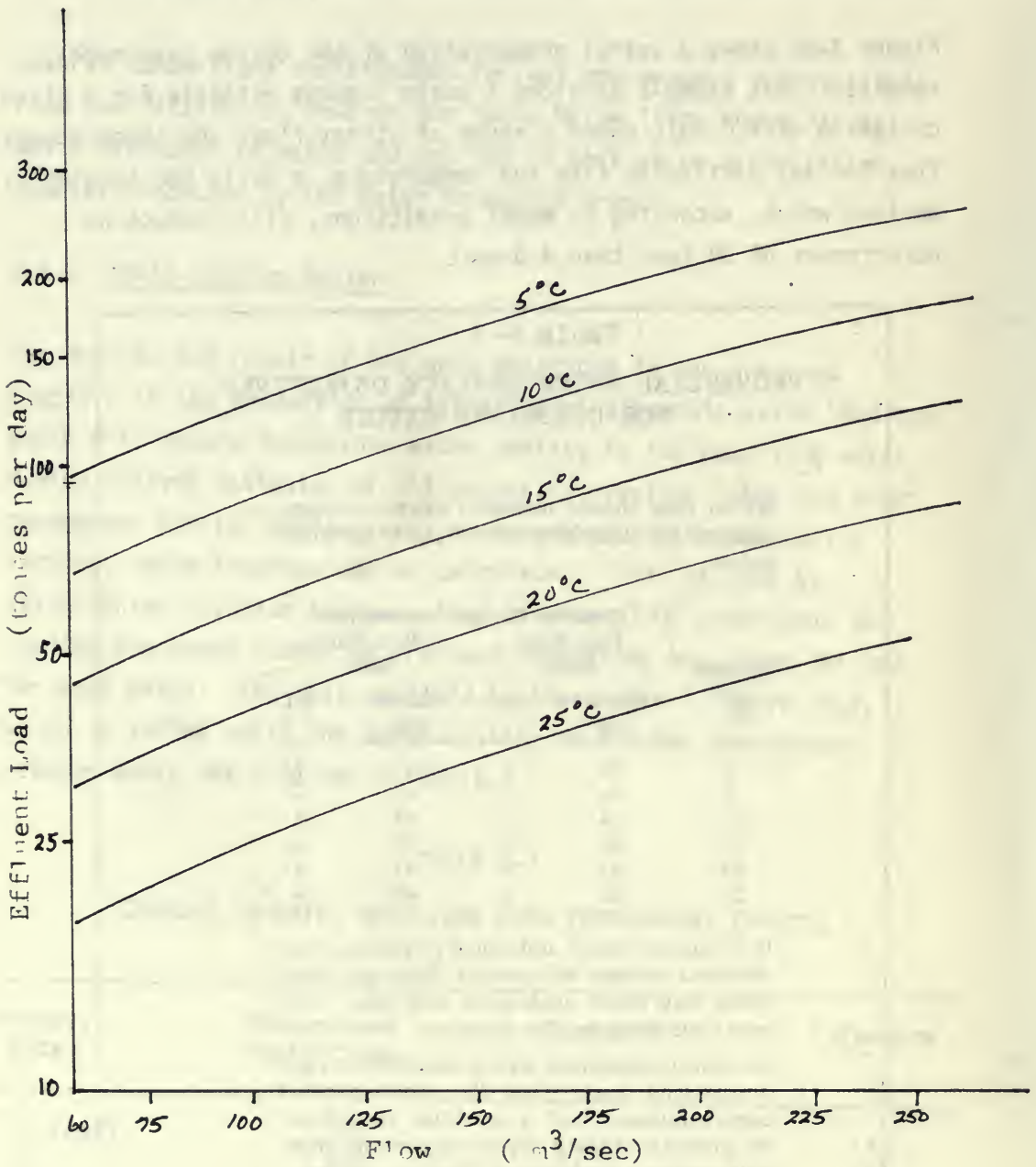
Temperature °C	Dissolved Oxygen Concentration Cold Water Biota		Warm Water Biota	
	% Saturation	mg/L	% Saturation	mg/L
0	54	8	47	7
5	54	7	47	6
10	54	6	47	5
15	54	6	47	5
20	57	5	47	4
25	63	5	48	4

In situations where additional physical and/or chemical stresses are present these minimum levels may prove inadequate and more stringent Objectives may be necessary.

In some hypolimnetic waters, dissolved oxygen is naturally lower than the above-specified concentrations. Such a condition should not be altered by adding oxygen demanding materials causing a depletion of dissolved oxygen.

Figure 4 - 20

PREDICTED WASTE LOADING AT
SPECIFIED FLOW AND TEMPERATURE TO
ACHIEVE A D.O. OBJECTIVE OF 4.0 mg/L



REFERENCES - CHAPTER 4

1. Gaudy, A. F., K. Komolrit, R. Follett, D. Kincannon, D. Modessitt; 1967; "Methods for Evaluating the First Order Constants k, and L for BOD Exertion"; Oklahoma State U., CWRE; Stillwater, Oklahoma.

2. Covar, A. P.; 1976; "Selecting the Proper Reaeration Coefficient for Use in Water Quality Models"; Proc. Envir. Modelling and Simulation; U.S. E.P.A. 600/9-76-016.

3. Weatherbe, D. G.; 1977; "Dissolved Oxygen Modelling of the Speed River, Ontario; Masters thesis; U. Of Waterloo, Dept. of Civil Engineering; Waterloo, Ont.

4. Weatherbe, D. G., T. P. H. Gowda; 1976; unpublished information re Grand River Basin waste assimilation studies; Water Modelling Section, Water Resources Branch, Ontario Ministry of the Environment; Toronto, Ontario, Canada.

5. Eckenfelder Jr., W. W.; 1970; Water Quality Engineering for Practicing Engineers; Barnes and Noble Inc.; New York, N.Y.; p. 51.

6. Butts, T. A., V. Kothandaraman, R. Evans; 1973; "Practical Considerations for Assessing the Waste Assimilative Capacity of Illinois Streams"; Circular 110; Illinois State Water Survey; Urban, Ill., U.S.A.

7. Ontario Ministry of the Environment; 1978; "Water Management - Goals, Policies, Objectives and Implementation Procedures of the Ministry of the Environment", Toronto, Ontario, Canada.

APPENDIX I

LOW FLOW FREQUENCY ANALYSIS FOR WASTE ASSIMILATION STUDIES

L.A. Logan

LOW FLOW FREQUENCY ANALYSIS

for

WASTE ASSIMILATION STUDIES

Prepared by:

L.A. Logan
Water Modelling Section
Water Resources Branch
Ministry of the Environment

March, 1977

STREAM WATER QUALITY AND WASTE ASSIMILATION

SEMINAR – March 10 and 11, 1977

LOW FLOW FREQUENCY ANALYSIS FOR WASTE ASSIMILATION STUDIES

by

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for

WASTE ASSIMILATION STUDIES

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1. INTRODUCTION

A water quantity flow model forms the basis of any stream water quality model. This is because the quantity of flow has major influences over the dynamic properties of the associated quality constituents such as influencing the reareation rate and dilution of effluents. Therefore, a proper hydrologic analysis of the available streamflow records is indispensable to wastewater assimilation studies.

The magnitude, duration and frequency of low flows are among the most critical criteria for this water quality modeling problem. Magnitude, duration and short term fluctuations of low flows are essential to the dynamic water quality modeling problem; while magnitude and frequency, in terms of the risk of exceeding the most severe low flow conditions are usually sufficient for the steady-state water quality models. This report dealt only with the hydrologic analysis for the steady-state water quality models. This entails the determination of drought flows (flows for an extended period of dryness) for a given period of the year and their recurrence interval in relation to given probability levels. If historical streamflow records are available the analytical procedures are straightforward. However, if there is inadequate, incomplete or lack

of data, a suitable data augmentation procedure such as flow correlation or data synthesis by parametric or statistical simulation models may be used to supplement and provide the necessary information for the analysis.

This treatment dealt with a brief review for the choice of a proper probability model for the data sequence and relates to practical application aspects of the analytical methods. The methods are applied to annual series and extended to a case of drought risk evaluation for partial-duration series.

Further details on the theory of drought flows and extreme-value probability analysis can be found elsewhere (Gumbel, 1963, Matalas, 1963, Fiering, 1964).

2. DROUGHT FLOW DESIGN FACTORS

Several basic design factors should be considered in the analysis of drought flows for water quality studies. Some of the most important criteria to be considered are:

a) Sources of data

b) Length and quality of historical records

c) Randomness and persitence in data sequence

d) Seasonality (non-stationarity of events)

e) Sampling (averaging) interval

f) Frequency (probability) distribution of events

g) Duration of drought

2.1 SOURCES OF DATA

The source of data is very important with regards to the placing of confidence and reliability in the analysis. For Ontario problems, the main source of streamflow data are:

- Water Survey of Canada, Environment Canada (all federal stations):

  1964-76, "Surface Water Data", Ontario 19xx

  1915-64, "Water Resources Paper Series for the St. Lawrence and S. Hudson Bay Drainage Region".

- Water Resources Branch, Ontario Ministry of Environment:

  1913-75 Hydrologic Information System (include all federal stations in Ontario, plus secondary stations monitored by the MOE).

Secondary sources of data:

- Conservation Authorities Branch
- Ontario Hydro-electric Commission
- MOE special studies
- Universities special studies.

Data obtained from sources outside of a qualified government agency or institution should be carefully screened for quality control before use.

2.2 LENGTH AND QUALITY OF HISTORICAL RECORDS

The drought flow with a 20-year return period (95% probability of non-exceeding the severity of the specified low flow) is the commonly used design flow in Ontario. In this case, an acceptable record length should be about 20 years (e.g. 2GA015, fig. 1). Smaller sample sizes such as 8-10 years may be used if the low flow event sequence fits a well defined frequency distribution (e.g. 2GA014, fig. 1). The use of very small sample sizes should be avoided or used with caution and only as preliminary estimates.

All available records for a consecutive period of unregulated flow should be utilized in the analysis. Regulated flow should be appropriately adjusted for effects of dam operation or other water supply diversions before use. If a well established pattern of flow regulation exist over a considerable number of years, these flows may be analyzed separately and interpreted likewise (e.g. 2GB001, fig. 8.1). However, if there is evidence of significant distortions in the frequency curves (e.g. 2GA028, fig. 8.2), the analysis should be discarded, since no meaningful evaluation can be achieved.

Correlation procedures may be used to extend short records or to fill data voids. However, it should be ensured that the reliability of the statistical estimates of the extended records is as good or better than the observed records.

2.3 RANDOMNESS AND PERSISTENCE IN DATA SEQUENCE

It is usually assumed that the drought flow sequence comprise mutually independent random events from a known probability distribution. There is, however, some dependence between successive events (persistence) of stream-flow observations. This persistence may exist even within the annual sequence of low flow events. The degree of persistence (serial correlation) is usually greater for partial duration series. The extent of the degree of persistence may be determined from the significance of the lag-k serial correlation coefficient plot (e.g. fig. 10.1). In most cases, persistence in annual sequence of low flow events is usually not significant and is often neglected; however, otherwise, the persistence may be removed by a simple smoothing of the data sequence. In Ontario, a calendar year is usually a satisfactory choice of interval for the annual low flow sequence.

2.4 SEASONALITY (NON-STATIONARITY) OF EVENTS

Some Ontario streams experience distinct summer and winter low flow conditions. In some streams the summer

flows might be the more severe or vice-versa. It is therefore desirable to identify, a priori, the desired and more critical low flow periods. Most waste assimilation studies are usually related to the summer flow conditions. Therefore, analysis may be based on available records for May to November for southern Ontario rivers and for June to October for northern Ontario rivers. A separate analysis may be required for winter low flow conditions in cases where these flows can be shown to be the more critical.

Although the data sequence may have been selected for a given season some evidence of non-stationarity (the variance of the sequence of events vary with time) in the data may still be present. This condition might be suspected if no suitable smoothed probability curve can be fitted to the data (e.g. fig. 10.2). The proper identification and removal of non-stationarity in given data sequence involve advance time series analysis which is beyond the scope of this report. Anyway, it suffices to say that the data sequence can be made stationary by suitable differencing technique.

2.5 SAMPLING (AVERAGING) INTERVAL

In the analysis of the drought flows, the magnitude of the minimum flow during each calendar year is considered; that is, the average minimum flow for a given period (sampling interval) within the year. The averaging interval may be 1-, 3-, 7-, ... - 30-days, etc. depending on the nature and

quantity of the waste load and extent of minor flow regulations. Design flows based on 1- or 3-days average interval is usually more critical than 7- or 15-days average intervals. The differences in magnitude of these flows may be substantial for some streams and only marginal for other streams (e.g. figs. 6.1-6.3, 7). The 7-day average minimum flow is the most commonly used interval in Ontario. The 7-day average usually provide for a smoothing of any unusual low 1-day fluctuations. However, the choice of the averaging interval should depend on the physical nature of the problem and should coincide more closely with a reasonable passage time of waste load through critical reaches.

2.6 FREQUENCY (PROBABILITY) DISTRIBUTION OF EVENTS

The data sequence should be fitted to a suitable probability model before any interpolation or extrapolation of the flows and their associated risks are made. Several types of probability models are available, ranging from Distribution Free (e.g. fig. 1), Normal (e.g. fig. 2), Log-normal (e.g. fig. 3), Gumbel's Extremal (e.g. fig. 4) and Pearson Type-3 (e.g. fig. 4) models. Investigation by Matalas 1963, has shown that the Gumbel's and Pearson Type-3 distributions are more suitable for low flows. The differences between these models are not significant in many cases; while other cases may require the choice of the more suitable model (e.g. fig. 4). For partial

duration series, the recurrence interval and encounter events may be assumed to be of Poisson distribution while the magnitude of the hazard events assumed to be exponential.

2.7 DURATION OF DROUGHT

The severity of a drought is closely associated with its duration. The duration of drought flow is more critical for the dynamic modeling problems and the analysis usually require advance statistical procedures for a meaningful evaluation. However, flow duration based on mean daily flows may be used to make inferences regarding the percent of time within a given year (or, on the average over a given number of years) the severity of a drought flow may be exceeded (e.g. fig. 11). Flow duration curves are more useful for the determination of draught characteristics of a given river.

3. DATA ASSEMBLY

The compilation of the necessary data for the drought flow analysis may comprise historical records or generated synthetic data.

3.1 HISTORICAL RECORDS

The historical records for a particular site or stream may be acquired from the available sources of information (Section 2.1). The data should be checked and edited for possible errors, inconsistencies, missing records and omissions. This may be done by an examination of the hydrographs, comparison of hydrographs of adjacent streams, unusual flow events in relation to the prevailing climatological conditions and possible unrecorded periods of flow regulations.

Data voids may be filled by a simple flow/flow correlation and drainage area-ratio procedures.

3.2 DATA EXTENSION

The extending of a short period of record at a particular station may be accomplished by flow/flow or flow/precipitation correlation between concurrent record of streamflow and/or climatic records. For example, a long term streamflow station record may be used to extend the short term station record

$$Q_i^{(2)} = f(Q_i^{(1)}) \qquad (1)$$

where $Q_i^{(2)}$ and $Q_i^{(1)}$ denote flows at the short term and long term station, respectively, for the i^{th} month ($i = 1, \ldots, 12$)

and f(.) denote some empirical function (e.g. linear or non-linear regression) between the concurrent flow sequences.

Daily estimates may be derived with the approximate relationship:

$$\hat{Q}_{ij}^{(2)} = Q_{i}^{(2)} \frac{Q_{ij}^{(1)}}{\sum_{j=1}^{N} Q_{ij}^{(1)}} \qquad (2)$$

where $Q_{ij}^{(2)}$ denotes the estimate of the j^{th} day flow for the i^{th} month at station (2); $Q_{ij}^{(1)}$ denotes the observed j^{th} day flow for the i^{th} month at station (1) and $\Sigma Q_{ij}^{(1)}$ denotes the sum of the mean daily flows for the i^{th} month at station (1).

In order to ensure reliability in the estimates of the mean and variance of the extended record, based on linear regression, the following reliability criteria should be satisfied (Matalas and Jacobs, 1964):

$$|r| > \frac{1}{\sqrt{N_1 - 2}} \qquad (3)$$

where r denotes the correlation coefficient between the concurrent records at stations (1) and (2); and N_1 is the length of the short record;

$$r = b \cdot S_1/S_2 \qquad (4)$$

where b denotes the regression coefficient, S_1 and S_2 denote the standard deviation of the concurrent records for stations (1) and (2), respectively. Typical critical minimum values of r for given N_1 are:

N_1 :	10	15	20	25	30
r :	0.35	0.28	0.24	0.21	0.19

Alternative regression relationships may be derived for flow extension between the short term flow station and long term precipitation. Stochastic models (autoregressive or moving average) can also be employed for single-site data extension; however, this approach belongs to advance simulation studies and the technique should be used only if absolutely essential.

3.3 SYNTHETIC DATA

In cases where no data is available at a particular sub-basin, data may be synthetized with the use of regional, parametric or stochastic model simulation techniques.

3.3.1 Regulated Flow Adjustment

The historical record may be corrected for the effects of flow regulation by a suitable flow reconstruction and routing procedure. Flow reconstruction implies the accounting for the inflow, outflow and change-in-storage, relative to a reservoir or dam operations. Inflows include upstream river flow inputs, local inflows and direct precipitation input to the lake of the reservoir. Outflows include the reservoir releases, diversions and evaporation from the reservoir lake surface. The change-in-storage ΔS, is provided by the continuity equation:

$$\Delta S = I - Q \qquad (3.1)$$

If records of the daily inputs are avilable, the change-in-storage may be computed from changes in daily reservoir lake levels in accord with the reservoir elevation storage relationship; then estimates of the output, Q, can be provided by the continuity equation. Daily records of precipitation and potential evaporation will be required to account for these secondary inputs and outputs, respectively.

Subsequently, the flow estimates, Q, may be routed downstream to the point of interest according to the routing equation:

$$QL_t = B \cdot QL_{t-1} + A \cdot (Q_t + Q_{t-1})/2 \qquad (3.2)$$

where QL_t and QL_{t-1} denote the lagged flows at time t and (t-1) respectively. A and B are lagging coefficients given by:

$$A = 2/(2.DT + 1)$$

and

$$B = (2.DT - 1)/(2.DT + 1)$$

where DT = lag (hr.)/24.

If a suitable record of unregulated flows is available for a gauge upstream of the dam, then these flows may be lagged to the downstream point of interest (disregarding the dam). The lagged flows are then augmented by the drainage-area ratio to provide the required estimates of natural flows:

$$\hat{Q}_t = QL_t \cdot \frac{AR\ (2)}{AR\ (1)} \qquad (.3.3)$$

where $\hat{Q}_t$ denotes the required flow estimate and AR (1) and AR (2) denote the total drainage area relative to the upstream and downstream flow points respectively.

3.3.2 Regional Analysis

This approach involves correlating streamflow statistical parameters (mean, variance, correlation coefficient, etc.) with the physical parameters (drainage area, slope, shape factor, etc.) of the catchments within a specified region:

$$\bar{Q} = f(A, S_1, n, .., ..)$$
$$S^2 = f(A, S_1, n, .., ..) \quad (5)$$
$$r(k) = f(A, S_1, n, .., ..)$$

where $\bar{Q}$, S and r(k) denote mean, variance and serial correlation coefficient of lag-k, respectively; $f(A, S_1, n ..)$ denotes some functional of the catchment parameters: area A, slope S_1, shape factor, n, etc.

Once the set of empirical relationships, based on equation (5) is determined for a given region, estimates of $\bar{Q}$, S^2 and r(k) .., for a given sub-basin (gauged or ungauged can be calculated. Subsequently, flow estimates for given time periods (annual, monthly or daily) may be synthetized with the use of a suitable flow generator such as:

$$Q_i = \bar{Q}(1 - r(k)) + r(k)Q_{i-1} + S(1 - r^2(k))^{\frac{1}{2}}\eta i \quad ..(6)$$

where Q_i and Q_{i-1} denotes estimates of flows at the site for the i^{th} and $(i-1)^{th}$ period, respectively; and ηi denotes a random number satisfying a distribution with zero mean and unit variance. This, by itself, is a major study, details of which can be found elsewhere (Benson and Matalas, 1967).

3.3.3 Parametric Method

Parametric watershed models can also be used to provide synthetic data at ungauged sites. This approach involves the calibration of the model at several sites within a given region. Correlation analysis is used to establish empirical relationships between select group of optimized model parameters and physical basin parameters. Once these empirical relationships are established, estimates of the model parameters in the designated ungauged area can be determined and subsequently the model can be used to generate flows with the use of precipitation or other input of meteorological variables that the model may require. This is also a major study which would require separate treatment.

3.3.4 Stochastic Method

This method may comprise a substantial portion of the regional analysis and data extension procedures. The method may involve the use of autoregressive and moving-average statistical models of specified order to generate annual sequence of synthetic data. Alternatively, the Thomas-Fiering method of month-to-month correlation structure for non-stationary sequence may be employed to generate synthetic sequences of monthly flows. Both methods, however, require the use of a random number generator to produce mutually independent events with zero mean and unit variance, which forms a part of the model structure.

4. DATA ANALYSIS

The analysis of the drought flow data is treated on the basis of selecting an appropriate probability model for the data sequence. This is a pre-requisite to calculating the design flow. The concept of distribution free method, Normal, Log-normal, Gumbel's Extremal and Pearson Type-3 distributions are briefly reviewed and compared. Poisson and exponential distributions are considered for partial-duration sequence.

4.1 DISTRIBUTION-FREE METHOD

The distribution-free method as applied here, implies that no attempt is made to fit the data to a theoretical distribution. A smoothed empirical curve to the cumulative frequency of the ordered data is used to represent the distribution. The method is simple:

a) order the data sequence, (x_i), according to decreasing order of severity (the smallest low flow given the highest rank, and the largest low flow the lowest rank):

$$x_1 < x_2 < \ldots x_k < x_{k+1} \ldots < \ldots \quad (7)$$

(b) compute the comulative probability, P_k,

$$P_k = \frac{k}{N+1} \quad (8)$$

(c) plot P_k vs x_k, for ranks k = 1, .., n

(d) fit, by eye, a smooth curve through the plotted points

e) compute estimates of the recurrence interval T_r by interpolation

$$T_k = 1/P_k \qquad (9)$$

This method cannot be used satisfactorily for extrapolating design flows outside of the range of the data used in the analysis (e.g. fig. 1).

4.2 NORMAL DISTRIBUTION

The data sequence may be regarded as normally distributed if the probability density function is Gaussian ('bell-shaped') given by:

$$f(x) = \frac{1}{\sigma\sqrt{2\pi}} \exp\left(-\frac{(x-m)^2}{2\sigma^2}\right) \qquad (10)$$

where $f(x)$ is the density function of the variable x; m and σ^2 denote the mean and variance of the distribution, respectively. These parameters (m, σ^2) are sufficient statistics to describe the normal distribution. The unbiased estimates $\bar{x}$ of m and S^2 of σ^2 are usually, determined from observed samples of size N:

$$\bar{x} = \frac{1}{N} \sum_{i=1}^{N} x_i \qquad (11)$$

$$S^2 = \frac{1}{N-1} \Sigma(x_i - \bar{x})^2 \qquad (12)$$

For practical purposes, there are a number of simple approaches which can be used to test for normality. The data sequence may be regarded as approximately normal if, the cumulative

frequency plots as a straight line on arithmetic probability paper (e.g. fig. 2), and if the coefficient of skew, g_1, and Kurtosis, g_2 , are zero and three (3), respectively:

- skewness: $$g_1 = \frac{N^2}{(N-1)(N-2)} M_3/S^3 \quad (13)$$

- Kurtosis: $$g_2 = \frac{N^2 (N+1)}{(N-1)(N-2)(N-3)} M_4/S^4 - \frac{3(N-1)^2}{(N-2)(N-3)} \quad (14)$$

where $$M_3 = \frac{1}{N} \sum_{i=1}^{N} (x_i - \bar{x})^3 \quad (15)$$

and $$M_4 = \frac{1}{N} \sum_{i=1}^{N} (x_i - \bar{x})^4 \quad (16)$$

Because of sampling errors in the observations $g_1 \neq o$ and $g_2 \neq 3$. However, the normal probability model may be regarded as satisfactory if:

$$g_1 < t_\alpha \left(\frac{6}{(N+3)}\right)^{\frac{1}{2}} \quad (17)$$

and $$g_2 < t_\alpha \left(\frac{24}{(N+5)}\right)^{\frac{1}{2}} \quad (18)$$

where t_α represent the value of the standard normal deviate corresponding to given probability levels (e.g. t_α = 1.64, 1.96 and 2.58 for confidence level 99%, 95% and 90%, respectively).

Application of the method is also straightfoward:

a) perform procedures (a) and (b) of the distribution free approach

b) plot P_k vs x_k, k=1, ..., N on arithmetic probability paper

c) compute $\bar{x}$, s, g_1 and g_2

d) plot $(\bar{x} + s)$ at $P_r = 84\%$, $(\bar{x} - s)$ at $P_r = 16\%$ and $\bar{x}$ at $P_r = 50\%$ and fit a straight line through these points,

e) visually check closeness of fit of the curve to the observed data points,

f) test g_1 and g_2 according to the inequalities (17) and (18) for specified probability levels. If these conditions are not violated, then the normal distribution can be regarded as an acceptable approximation for the data sequence.

The recurrence interval (exceedance probability) can be interpolated or extrapolated from this curve. Caution should be exercised when extrapolating, especially when the very extreme data points deviate significantly from the normal curve (e.g. fig. 2).

4.3 LOG-NORMAL DISTRIBUTION

If the data sequence cannot be adequately represented by a normal distribution, a log-normal distribution may be assumed; that is the probability density function:

$$f_{ln}(x) = \frac{1}{x\sigma\sqrt{2\pi}} \exp\left(-\frac{1}{2\sigma^2}(\ln x - m)^2\right) \qquad (19)$$

The procedure may involve a simple log-transformation of the drought flow sequence and repeating the same operations done for the normal distribution of the previous section. Alternatively, the data may be regarded as approximately log-normal if a plot of the raw data on log-probability paper defines a straight line (e.g. fig. 3). Drought flows are subjected to a lower limit of zero and because of the usually small sample size available it is often desirable to select a

probability model with less than three parameters. Although the log-normal model satisfies this condition, it is rarely satisfactory for drought flow events. However, if these probability models are not adequate, the Gumbel's extremal or Pearson Type-3 may be used.

4.4 GUMBEL'S EXTREMAL DISTRIBUTION

The Gumbel's limited distribution of smallest values is ideally suited for drought flow analysis. The cumulative distribution, which is more suitable for plotting, is given by:

$$F(x)=\left(1-\exp\left(-\left(\frac{x-\varepsilon}{\mu-\varepsilon}\right)^{\alpha}\right)\right) \qquad (20)$$

$$\varepsilon < x < \infty$$

where $F(x)$ is the cumulative distribution of x ; μ and ε denote the characteristic drought flow (the value of the flow with an exceedance probability of 63.3% or a recurrence interval of 1.58 years) and the lower limit of the drought flow respectively; and α is a scale parameter (analogous to standard deviation for a normal distribution).

In order to fit equation (20) to a set of drought flows, it is necessary to estimate the parameters u, ε and α . These parameters are functions of the statistical parameters according to the following relationships:

$$\mu = \bar{x} + A_{\alpha} S \qquad (21)$$

$$\varepsilon = \mu - B_{\alpha} S \qquad (22)$$

where A_{α} and B_{α} are coefficients which can be determined from standard tables (Table 2a) for given estimates of the

skew coefficient. For ease of application, the augument of the exponent in equation (20) can be more convienently expressed as the log-transformation:

$$\log (x - \varepsilon) = \log (\mu - \varepsilon) + {}^{1}/_{\alpha} \log y \qquad (23a)$$

where $$y = \left(\frac{x - \varepsilon}{\mu - \varepsilon}\right)^{\alpha} \qquad (23b)$$

The following is a brief outline of the steps in the application of the method.

a) order (rank) the low flow sequence in decreasing order of severity and compute the distribution-free plotting positions (equations (7), (8), (9)).

b) compute sample estimates $\bar{x}$, S, M_3 and g_1 (equations (11), (12), (13) and (15)).

c) estimate the value of the parameters ${}^{1}/_{\alpha}$, A_{α}, and B_{α}, for the computed estimate g_1 (Table 2a of the Appendix).

d) compute estimates of μ, and ε (equations (21), (22)).

e) for specified values of F(x) compute y (equation (23b).

f) use the estimates μ, ε, y and ${}^{1}/_{\alpha}$ in equation (23a) to estimate the corresponding value of the flow x,

g) plot x vs F(x) on logarithmic extremal probability paper and fit a smoothed curve through these points; then check for closeness of fit with observed data points (e.g. figure 4).

If the theoretical curve is a straight line, then $\varepsilon = o$; if the curve convex upward, then $\varepsilon > o$; if the curve concave downward, then $\varepsilon < o$. However, for the drought flow sequence, the lower limit ε should be greater than zero - small negative vlaues for ε may be assumed to be zero. If these conditions are not satisfied then the Gumbel's distribution is not a satisfactory probability for the data sequence.

4.5 PEARSON TYPE-3 ASYMPTOTIC DISTRIBUTION

The Pearson Type-3 distribution is often a suitable alternative to the Gumbal's distribution. The cumulative distribution can be fitted according to the model:

$$x = \bar{x} + ks \tag{24}$$

where k denotes a frequency factor for given values of the probability F(x). The values of k can be determined in relation to the percentile values of the Chi-square distribution (χ^2) with a given degree of freedom:

$$k = \chi^2(F,\nu)/2C - C \tag{25}$$

where $\chi^2(F, \nu)$ is the frequency value for χ^2 with given degrees of freedom ν,

$$\nu = 8/g_2^1 \tag{26}$$

and C is a constant given by

$$C = 2/g_1 \tag{27}$$

The model can be fitted by performing the following operations:

a) compute the sample estimates: $\bar{x}$, S, M_3, g_1, ν and C (equations (11), (12), (13), (15), (26), (27),

b) use pre-assigned values of F(x) to determine values of $\chi^2(F,\nu)$ for the degrees of freedom ν (use standard χ^2-table - Table 3 of the Appendix), or

c) for given values of F(x) and the estimate of g_1 determine estimates of k with the use of Table 2b of the Appendix,

d) compute value of x using estimates of $\bar{x}$, S and the k-values (equation (24)),

e) plot x vs F(x) on logarithmic extremal probability paper; then check for closeness of fit with observed data points (e.g. (Fig. 4).

4.6 POISSON DISTRIBUTION: PARTIAL-DURATION SEQUENCE

In certain studies, it may be necessary to impose an upper limit on the design flow relative to an appropriate level of risk, over a known or established life of the project. That is, find a method to determine the likelihood of a drought flow event exceeding the severity of the design flow during the life of the project. A Partial-Duration analysis may be a suitable analytical procedure for the historical low flow events. This method will be briefly discussed starting with the following useful definitions:

i) hazard event - is the exceedance event ; that is, the value of a flow event exceeding the severity of the design flow,

ii) waiting time distribution - is the probability that the waiting time from some arbitrarily selected point in time to the occurrence of the first exceedance event is less than or equal to some numerical value,

iii) encounter probability - is the probability that an exceedance event will occur during the design period,

iv) expected recurrence interval - is the expected value of the waiting time distribution from some time, t_o, to the occurrence of the first exceedance event.

The Poisson distribution forms the basis for these probability relationships. A joint Poisson distribution is assumed for a waiting time interval and the number of exceedance events. In addition, an exponential distribution is assumed for the magnitude of the hazard events.

For drought flow events, the design equation may be expressed as (Shane and Lynn, 1964):

$$y = y_o - \gamma \log \left(- \frac{\lambda t}{\log(1-F_T(t))} \right) \qquad (28a)$$

or $$y = y_o - \gamma \log (\lambda E(t)) \qquad (28b)$$

or $$y = y_o - \gamma \log \left(- \frac{\lambda L}{\log (1-\bar{E})} \right) \qquad (28c)$$

where y and y_o denote the design flow and an assigned base flow (upper limit), respectively;

γ is a parameter associated with the exponential distribution,

λ is the expected rate of occurrence of the hazard events,

$F_T(t)$ and $E(t)$ denote the probability distribution of the waiting time, T, and the expected value of T respectively,

E denotes the encounter probability (the probability that an exceedance will occur within the design period); and L is the length of the design period in years.

In application, the procedure is also straightforward:

a) set an appropriate baseflow level and enumerate the hazard events

b) compute estimate $\hat{\lambda}$ of λ and $\hat{\gamma}$ of γ by:

$$\hat{\lambda} = N/n$$

$$\hat{\gamma} = Y_o - \bar{x}$$

where N and n denote the number of hazard events and the number of years of historical record and $\bar{x}$ is the mean of the hazard events given by:

$$\bar{x} = \sum_{i=1}^{N} x_i$$

c) compute estimate $\bar{T}_k$ of E(T) by the plotting formula $\bar{T}_k = n/k$

where k is the rank of the hazard event in drought flow sequence,

d) plot the sequence x_k vs $\bar{T}_k$ on simi-log paper and fit the model:

$$y = y_o - \hat{\gamma} \log (\hat{\lambda}\ \bar{T}_k).$$

The model may be regarded as satisfactory if the curve fits close to the observed data points (e.g. fig. 5.1).

In order to use this model in designing flows, the additional relationship between recurrence interval and encounter probability is required. Figure 5.2 shows a family of curves relating recurrence interval and encounter probability for various lengths of design period. For given encounter probability E, and design period L, the recurrence

interval $\bar{T}_k$ can be determined. With this $\bar{T}_k$, the design flow can be calculated with the empirical model of the partial-duration sequence.

5. APPLICATION AND DISCUSSION

In applying frequency analysis to drought flow data, the simpliest probability model should be used where possible. Resort to the more complex model only if the simple ones prove to be inadequate or if a thorough analysis is required of the flow data. The following discussion will deal with the results of some typical applications. Fig. 1 shows examples of the Distribution Free plots for three unregulated streams. Stations 2GA014 (Grand River at Marsville) and 2GA015 (Speed River at Guelph) display a satisfactory smoothed distribution curve; while Station 2GA010 (Nith near Canning) displays a less than satisfactory curve. Interpolation of drought flows for given risks are feasible, but they would be the least reliable for 2GA010.

Figures 2 and 3 show the Normal and Log-normal probability plots, respectively. The Normal probability model appears to be satisfactory for 2GA014, less than adequate for 2GA015 and poor for 2GA010. However, for the Log-normal plot there is some improvement in the fits for 2GA010 and 2GA015; and is distorted for 2GA014. The extreme value points for 2GA015, however, still deviates substantially from the theoretical curve.

Finally, Figure 4 shows the fitted Gumbel's and Pearson Type-3 distribution to these data points. In the case of 2GA010 and 2GA015, the models appear to be adequate, with only marginal differences between them. However, for the station 2GA014, only the Pearson Type-3 model gave a

reasonable fit; a fit with the Gumbel's model was not feasible because of a comparatively large negative lower limit ε. This may emphasize that an attempt to fit a more complex model to a data set where a simple model would suffice could only lead to distortions and possible erroneous design flow extrapolations. Table 1.1 shows the summary of the low flow statistics for the sample stations; while Table 1.2 summarizes the design flow for $7Q_{20}$ extrapolated for the different probability models. The sample computations for the Gumbel's and Pearson Type-3 fitted models are given in Tables 1a and 1b of the Appendix. Although the Gumbel's and Pearson Type-3 appear to be a good distribution for most low flow data, they can, however, be poor fits to some cases. This is dramatized in Figure 9 for 60-day average minimum low flow at 2GA014. The effect of varying n-days averaging intervals upon the frequency probability distributions are shown in Figures 6.1, 6.2 and 6.3. The averaging interval appears to be the most serious for small streams (Fig. 6.3) and least serious for larger streams (Figs. 6.2, 6.1) in terms of relative difference in design flow for various n-days averaging. However, the absolute difference in flow might be more important seeing that the smaller streams might not be capable of any waste assimilation. The design flow increases at a rate of 2.0 cfs per 7 days increase in averaging interval with recurrence interval of 20 year for 2GA010 and 2GA015 (Fig. 7). This implies that, for these

stations, at least, the n-Q_{20} design flow with averaging interval n = 1, 3, 5, 7 days would not be significantly different from one another in view of the sampling errors that are inherent in the original data. However, the level of risk (recurrence interval) is equally critical to all analysis and should be chosen according to established guidelines.

Figures 8.1 and 8.2 shows frequency curves of low flow data affected by regulation. The plots for Station 2GB001 (Fig. 8.1) implies that the dam operations (Shand and Conestogo) above Brantford, augmented the low flow frequency to a new higher level above the natural flow sequence of 1913-23. In other cases, the regulation may cause an incoherent break in the pattern of low flow frequency as demonstrated in Figure 8.2 for the Station 2GA028 below the Conestogo Dam. For example, the Pearson Type-3 model demonstrates a poor fit to the 1-day and 7-day average minimum flows. Encounters of this kind should point suspicion to possible defects in the data, if this was not known beforehand. In any case, the analysis should be discarded, or the raw data adjusted before use.

Figure 5.1 shows an example of a frequency plot for a partial-duration analysis of 7-day low flow for 2GA029 (Eramosa above Guelph). This analysis is very dependent on a proper choice of the reference base flow; and the assumptions concerning the probability distributions have to be more closely satisfied. Also, the model is more applicable to high flow sequence; but an attempt is made,

for this exercise, to fit an analogous model to the low flow sequence. However, unlike the annual series, this model works in relation with the joint distribution of encounter probability and recurrence interval of Figure 5.2 for the purpose of design. For example, with an encounter probability of 10% and design period of L = 2 years, the corresponding recurrence interval is 20 years, which corresponds to design flow of 12 cfs (Figure 5.1); that is, $7Q(2)_{20,.10}$ = 12 cfs. In application, both curves can be used to determine other design flows of given encounter probability and design period.

6. SUMMARY AND CONCLUSION

Examples of annual and partial-duration sequences of drought flows were evaluated by statistical frequency analysis. For the purpose of determining a reliable design flow emphasis has been placed on the importance of choosing the proper probability model for a particular drought flow sequence. For some data sequence, a simple distribution free method might be adequate for a determination of exceedance probability levels. Whereas in other cases, the more conventional probability model of the Gumbel's extremal or Pearson Type-3 distribution may be used. The choice of one model over the other is, however, still dependent on its applicability and goodness of fit to the particular data sequence. Unregulated flow data should always be used in the analysis, or the flow corrected for effects of regulation before use. Simple interstation flow/flow correlation may be used to extend short periods of records. However, the reliability of the statistical parameters of the extended record should be at least as good as that of the short record. Seeing that, in Ontario, the commonly used design low flow for waste assimilation is related to a risk of 95% non-exceedance (severity) probability, a suitable length of historical record for analysis would be at least 20 years. Shorter record lengths may be used but the results should be interpreted with caution. In cases where synthetic data are required, particularly for ungauged sites, a more elaborate procedure of data generation may be required. This usually requires

some form of regional statistical analysis, involving parametric or stochastic models. However, this approach is usually undertaken as a separate assignment, because of the large scale nature of the problem which is required in order to establish some degree of reliability in the synthetic data sequence.

The choice of averaging interval for low flows are usually more critical for small streams; but the differences in magnitude of design flow for a given risk may not be significant for small changes in the averaging interval. The choice of the averaging interval should, however, be according to established guidelines. Also, the choice of the exceedance probability is critical for all design cases, seeing that the risk relates to the chance of the project's failure rather than the magnitude of the design flow.

Some flexibility may be introduced into the design by considering low flow analysis based on partial duration sequences. However, the design is relegated to a given design period (life of the project) and to a joint risk of failure based on the encounter probability and the waiting time distribution.

Although occasional incidences of non-stationarity may exist in annual low flow sequence, this may not be significant enough to require special treatment; unlike persistence, which might be sufficiently significant in the data sequence so as to cause difficulty in fitting a suitable probability model.

7. REFERENCES

Benson, M.A. and Matalas, N.C. 1967 Synthetic Hydrology based on Regional Parameters. Water Res. Resources, Vol. 3(4): 931-935.

Chin, W.Q., 1967, Formulae and Tables for Computing and Plotting Drought Frequency Curves. Technical Bulletin No. 8, Inland Waters Branch, Dept. of Energy, Mines and Resources, Ottawa.

Fiering, M.B., 1964, A Markov Model for Low Flow Analysis. IASH Vol. 9(2) : 37-47.

Gumbel, E.J., 1963. Statistical Forecast of Droughts. IASH. Apr. 1963: 5-23.

Matalas, N.C., 1963. Probability Distribution of Low Flows - Statistical Studies in Hydrology, U.S.G.S. Profes. Paper 434-A.

Matalas, N.C. and Jacobs, B., 1964. A Correlation Procedure for Augmenting Hydrologic Data - Statistical Studies in Hydrology. U.S.G.S. Profes. Paper 434-E.

Shane, R.M. and Lynn, W.R., 1964. Mathematical Model for Flood Risk Evaluation, ASCE Jour. Hydro. Div. HY6 : 1-21.

U.S. Army Corps of Engineers, 1968. Daily Streamflow Simulation. The Hydrologic Engineering Center, Computer Program 24-C* - L235.

U.S. Army Corps of Engineers, 1971. Monthly Streamflow Simulation. The Hydrologic Engineering Center, Computer Program.

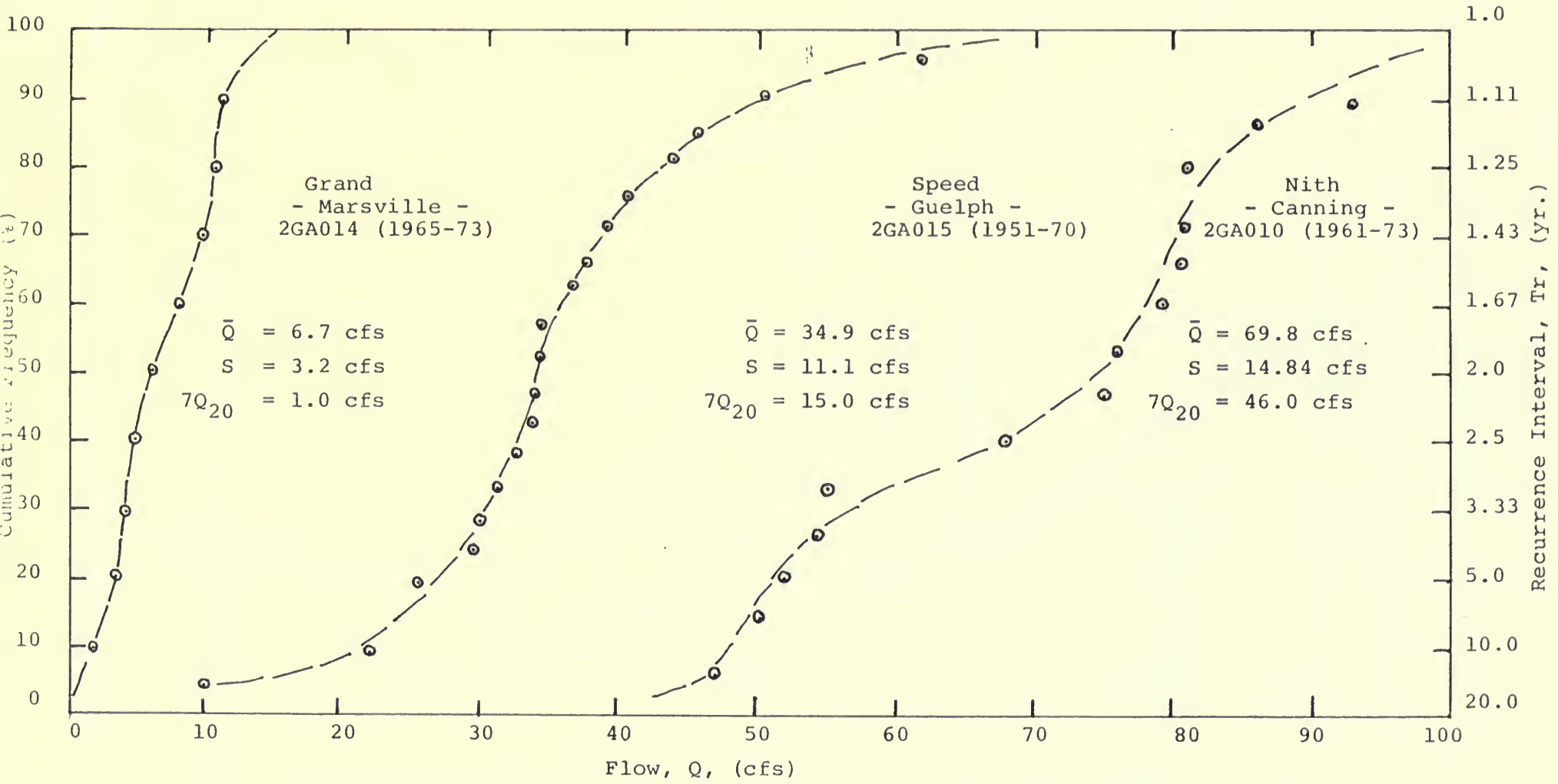

FIGURE 1: Distribution-Free Probability Plots of 7-Day Minimum Low Flows

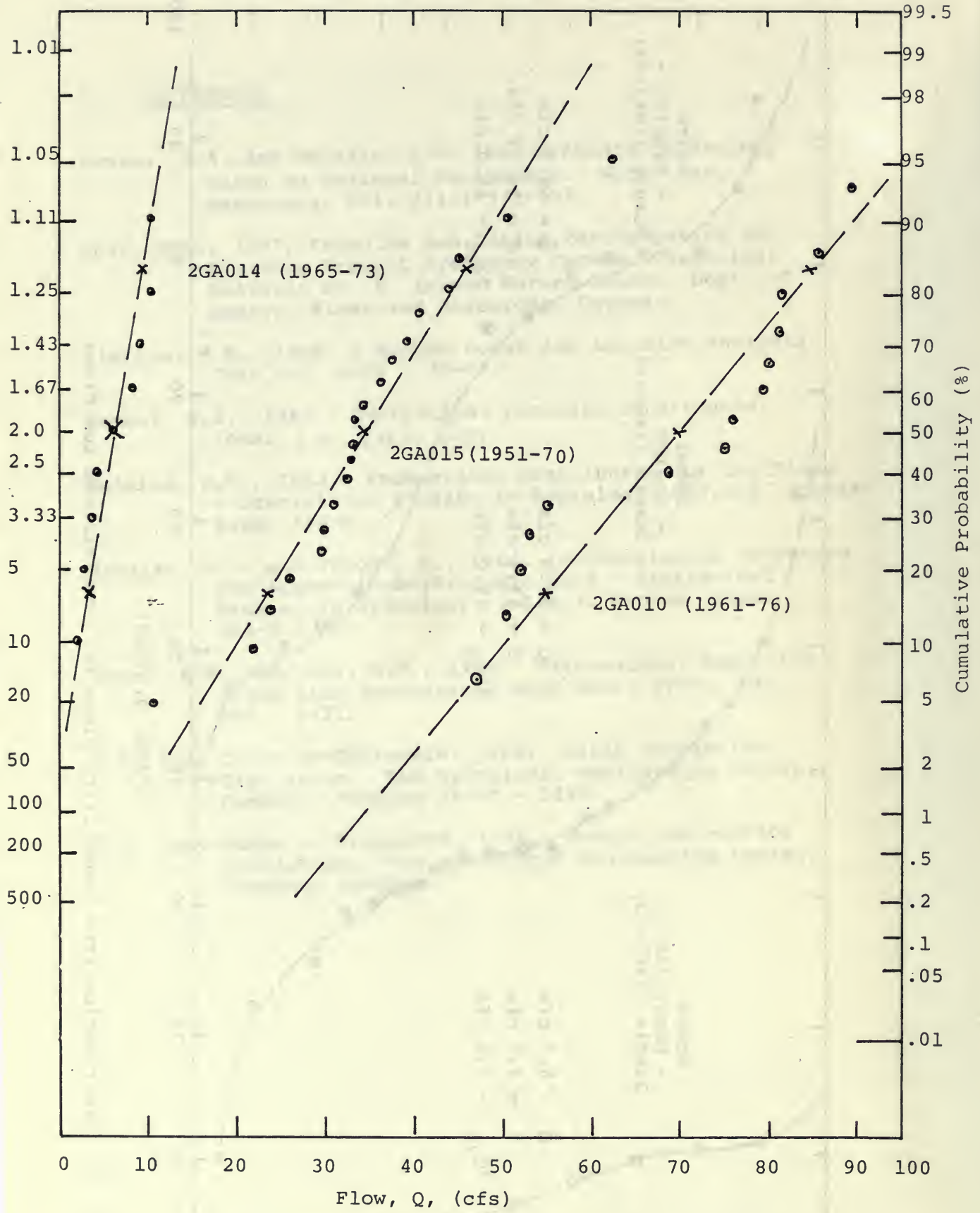

FIGURE 2: Cumulative Probability Distribution for 7-Day Average Minimum Flows: Normal Probability Plots

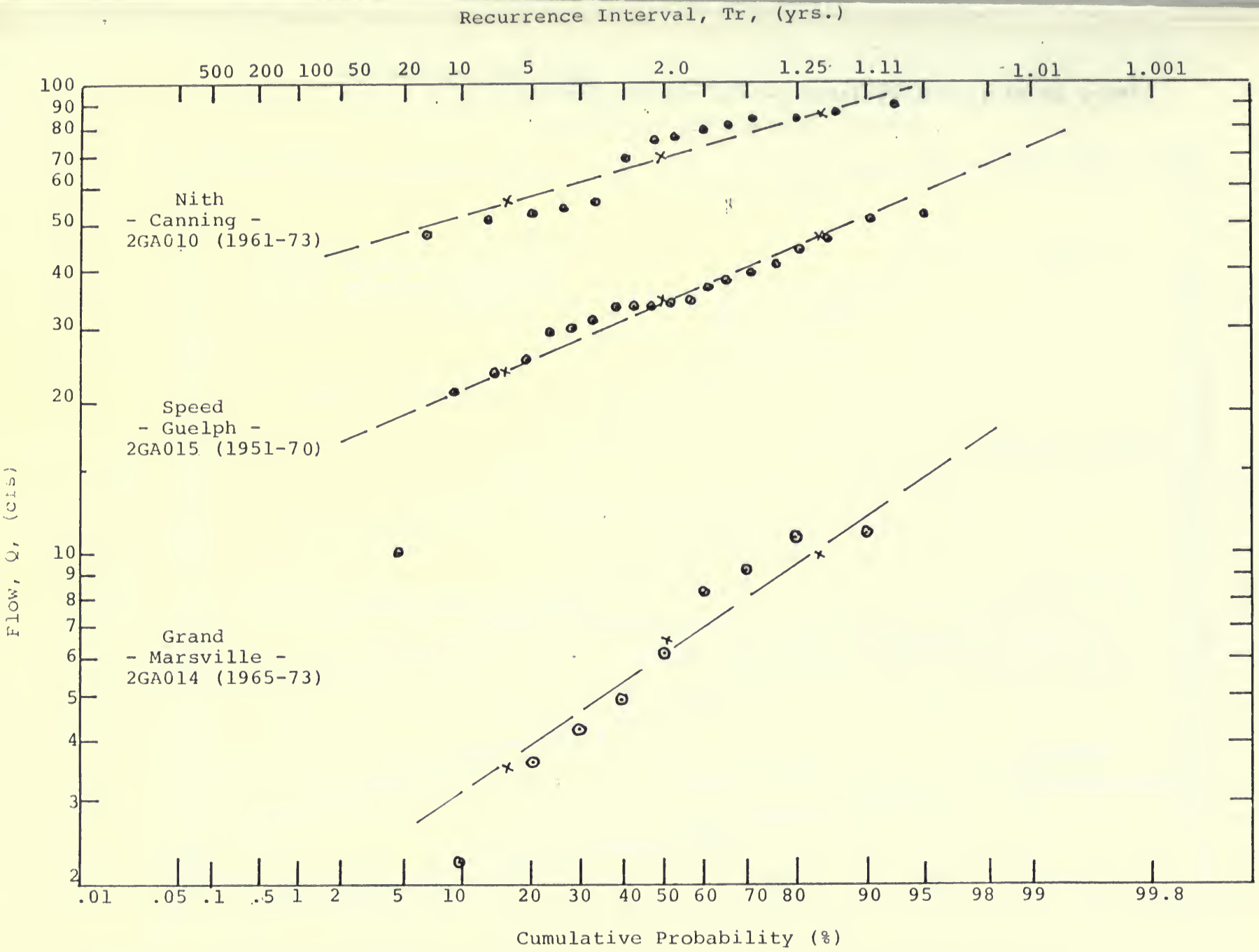

FIGURE 3: Cumulative Probability Distribution for 7-Day Average Minimum Flow: Log-Normal Probability Plot

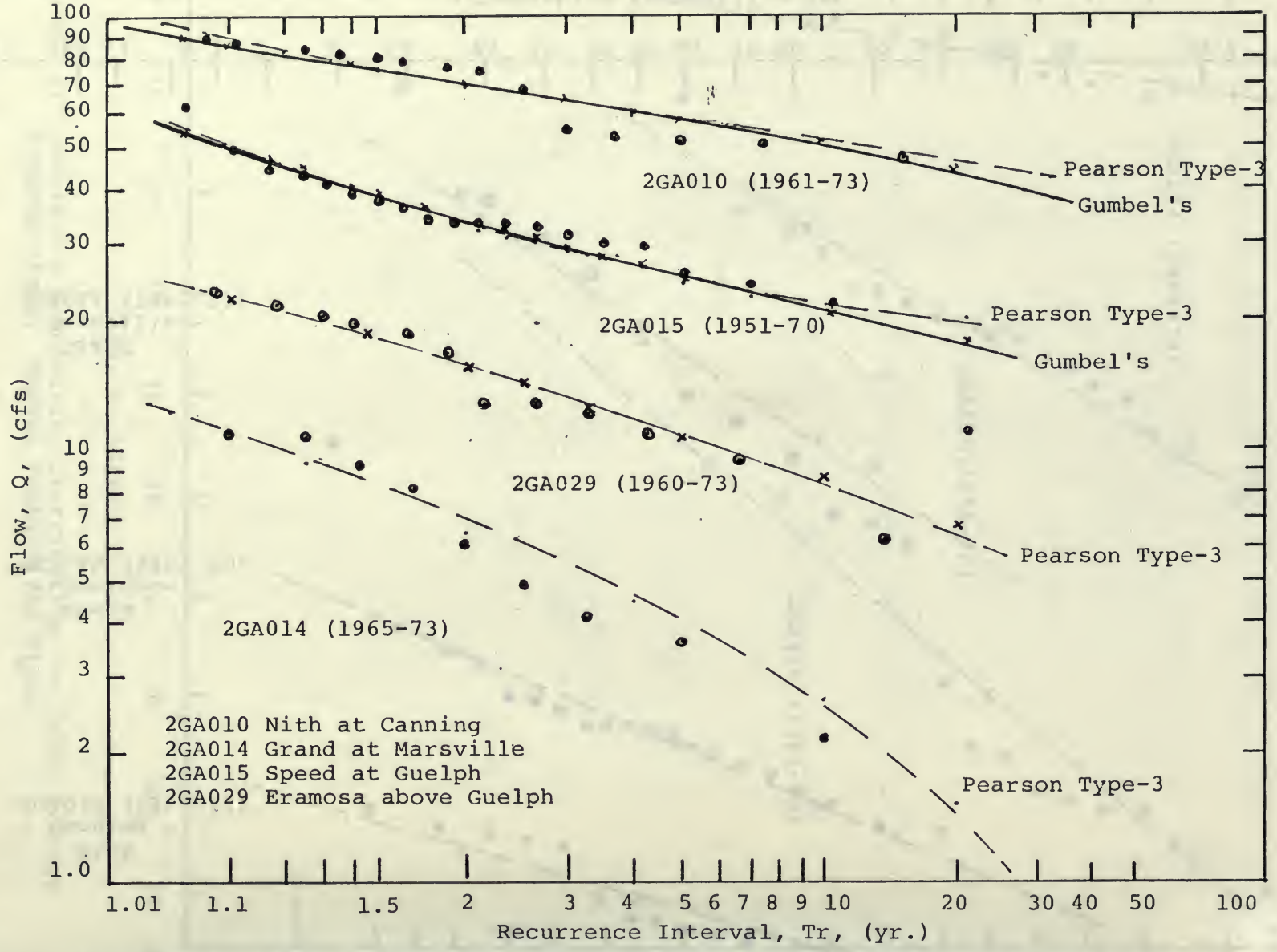

FIGURE 4: Gumbel's Extremal and Pearson Type-3 Distribution Plots of 7-Day Average Minimum Flows

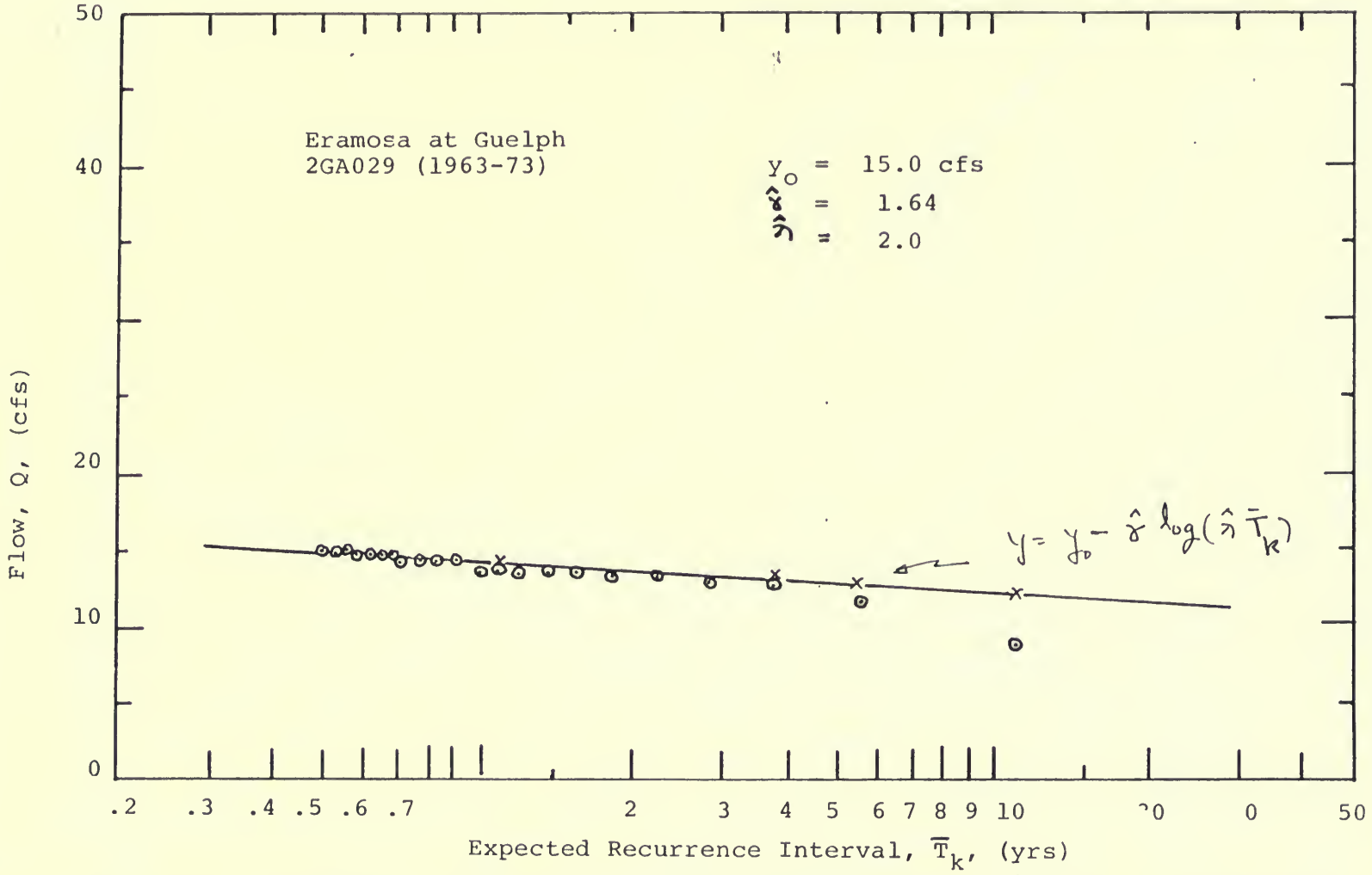

FIGURE 5.1: Frequency Distribution of Partial-Duration Series for 7-Day Average Minimum Flow

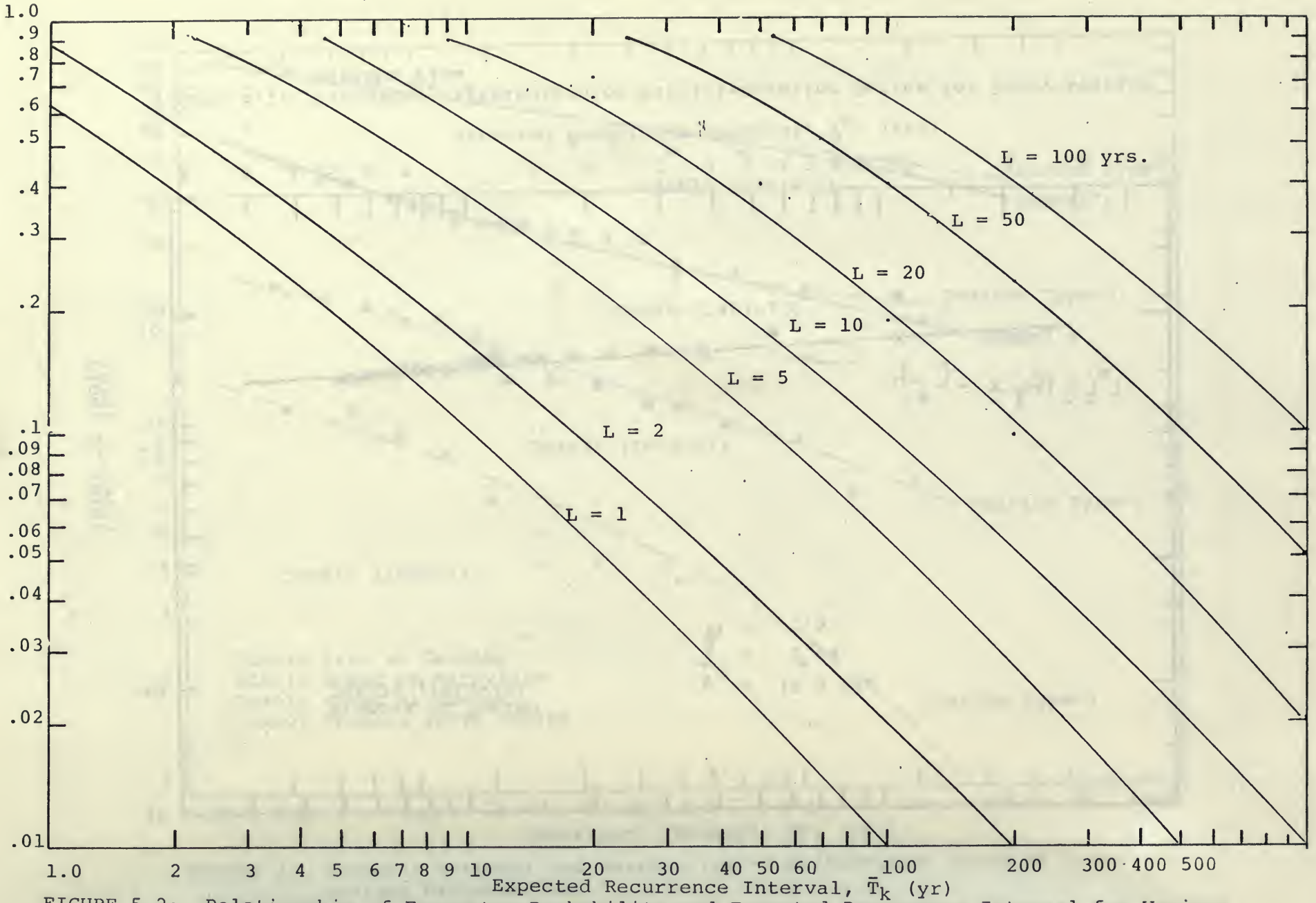

FIGURE 5.2: Relationship of Encounter Probability and Expected Recurrence Interval for Various Design Periods

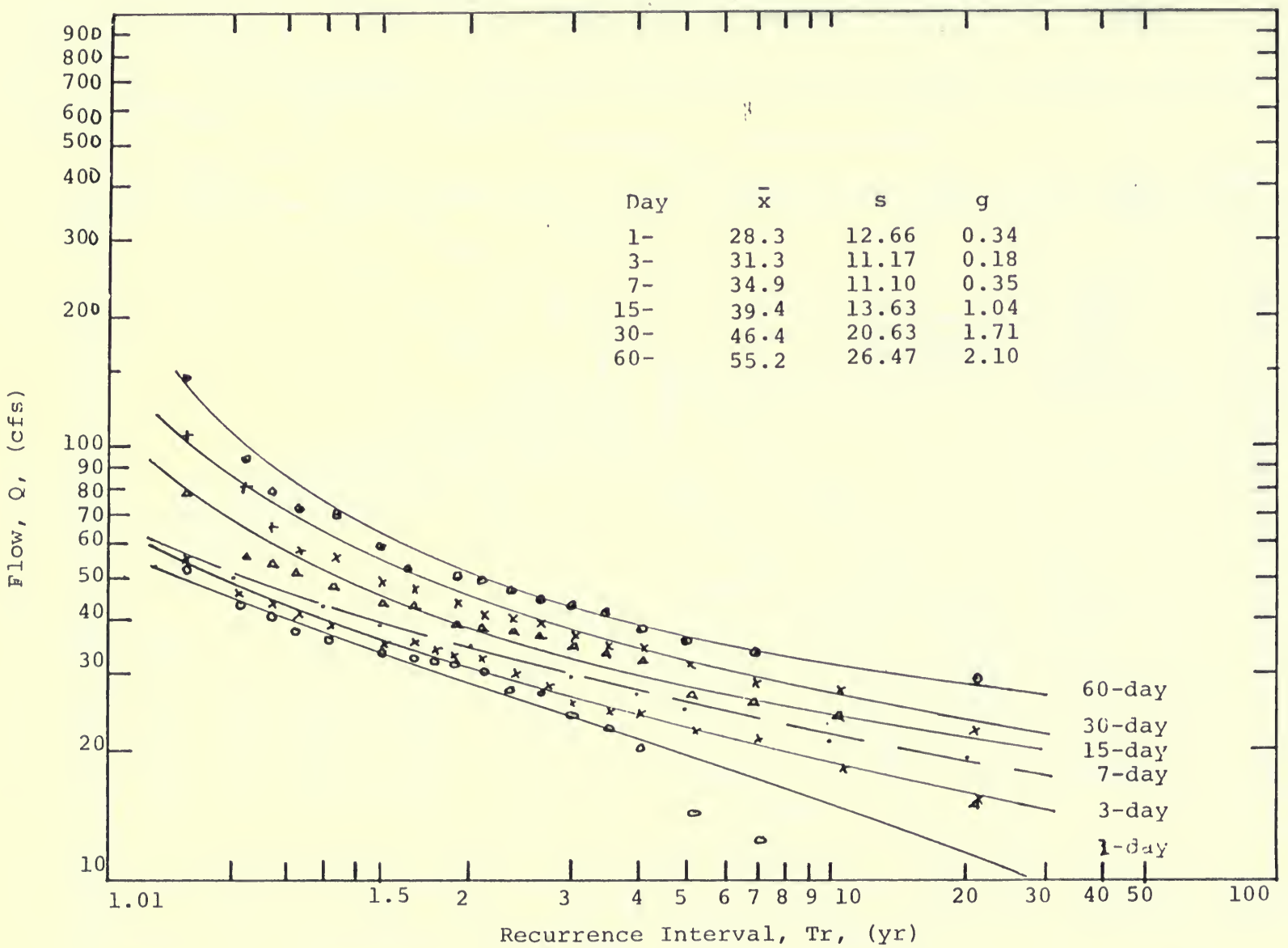

FIGURE 6.1: Drought Flow Frequency Curve for Varying n-Day Averaging Interval: Stn. 2GA015 (Speed River at Guelph)

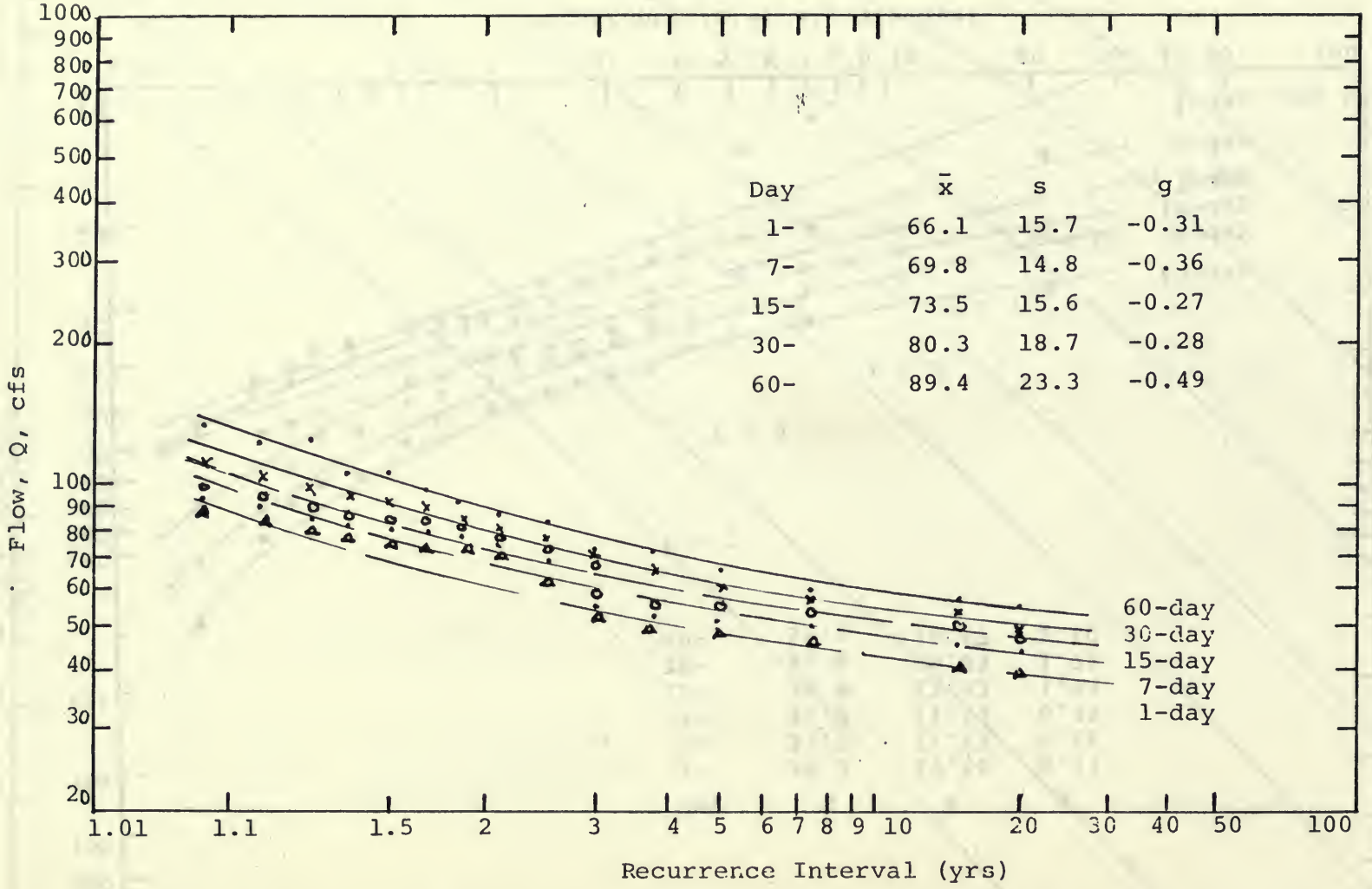

FIGURE 6.2: Drought Flow Frequency Curves for Varying n-Days Averaging Interval Stn. 2GA010 (Nith River at Paris)

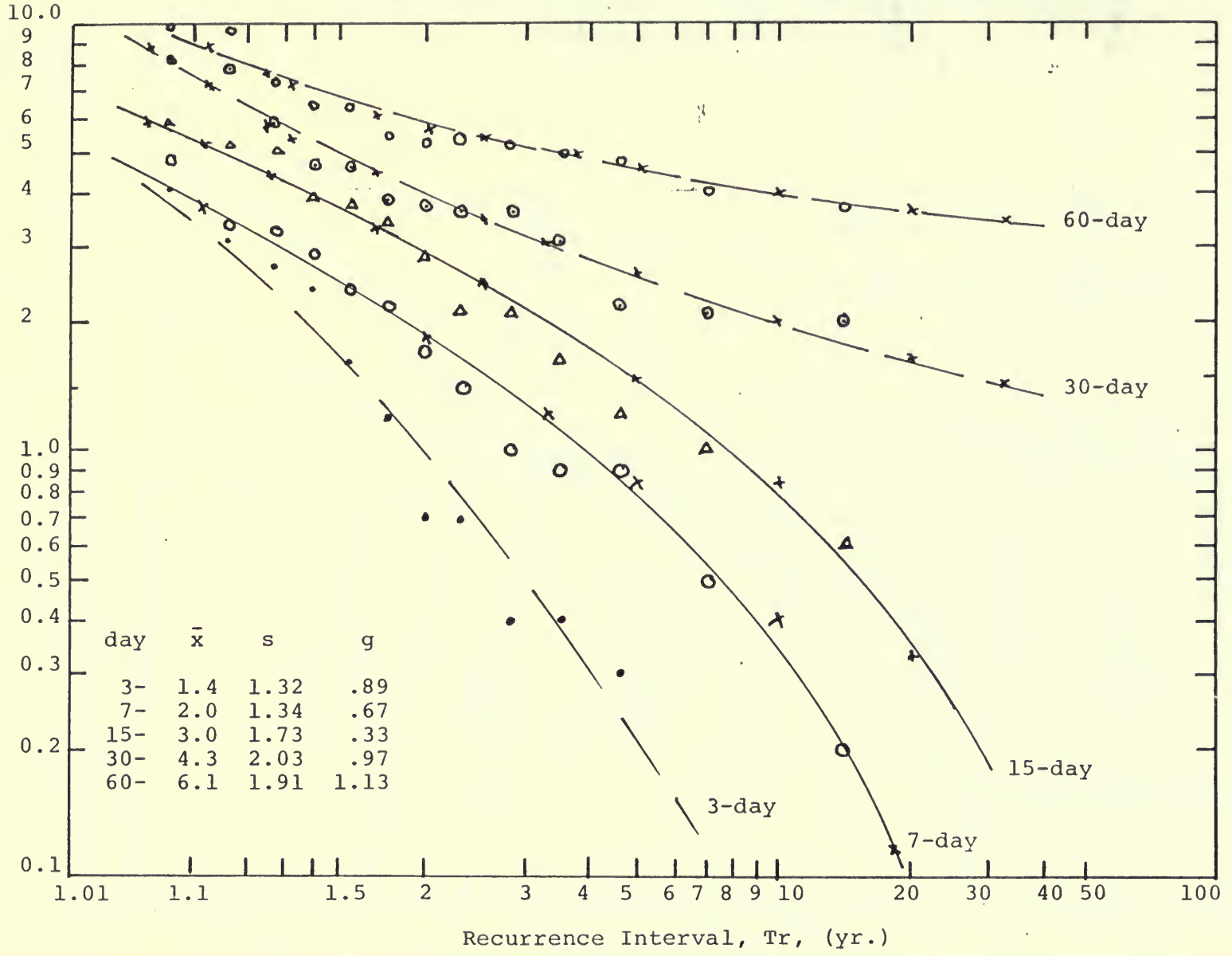

FIGURE 6.3: Drought Flow Frequenc Curves for Varying n-Days Averaging Interval: Station 2GBC10 (McKenzie Creek at Caledonia)

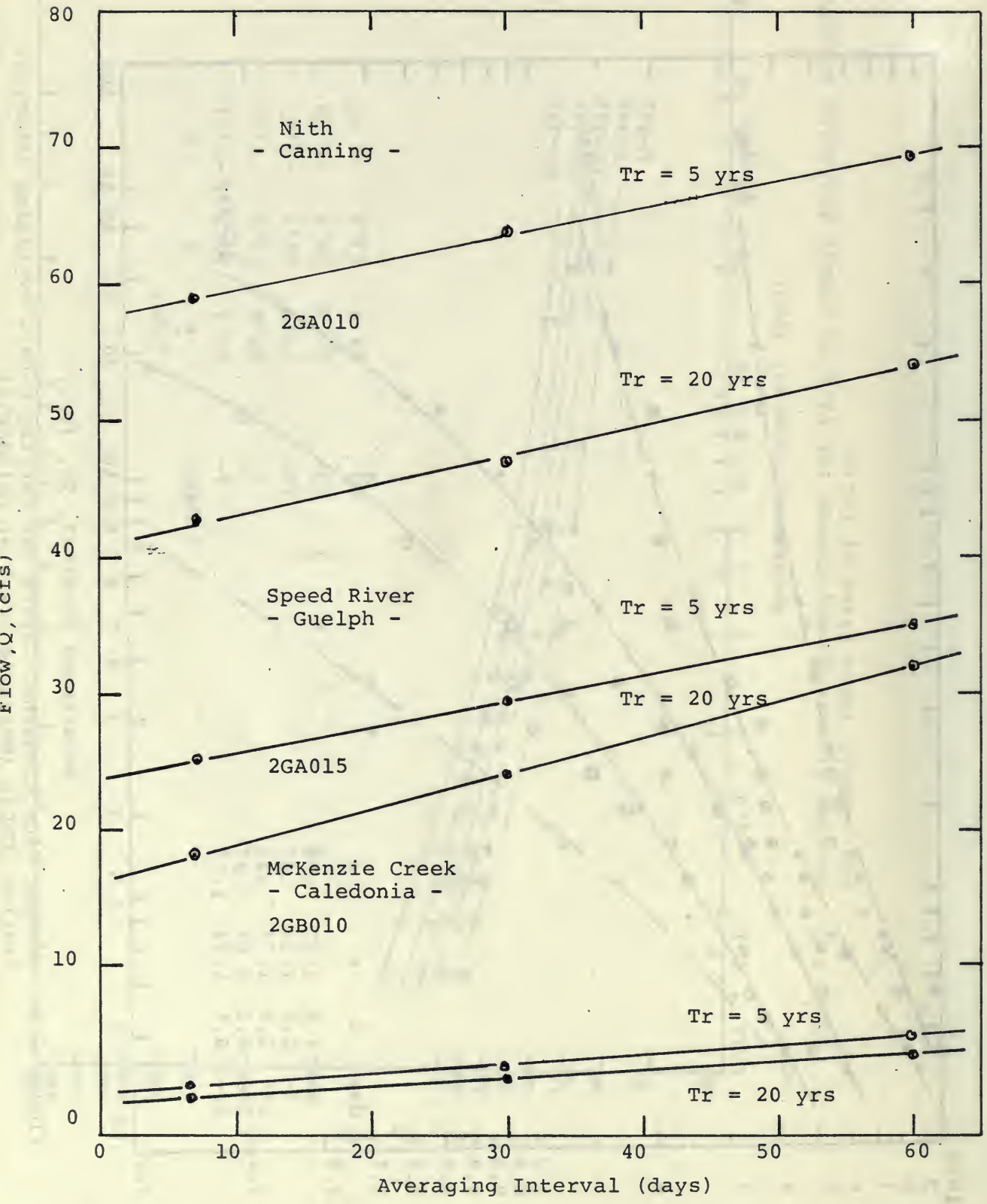

FIGURE 7: Drought Flows for Varying n-Day Averaging Intervals and Recurrence Intervals

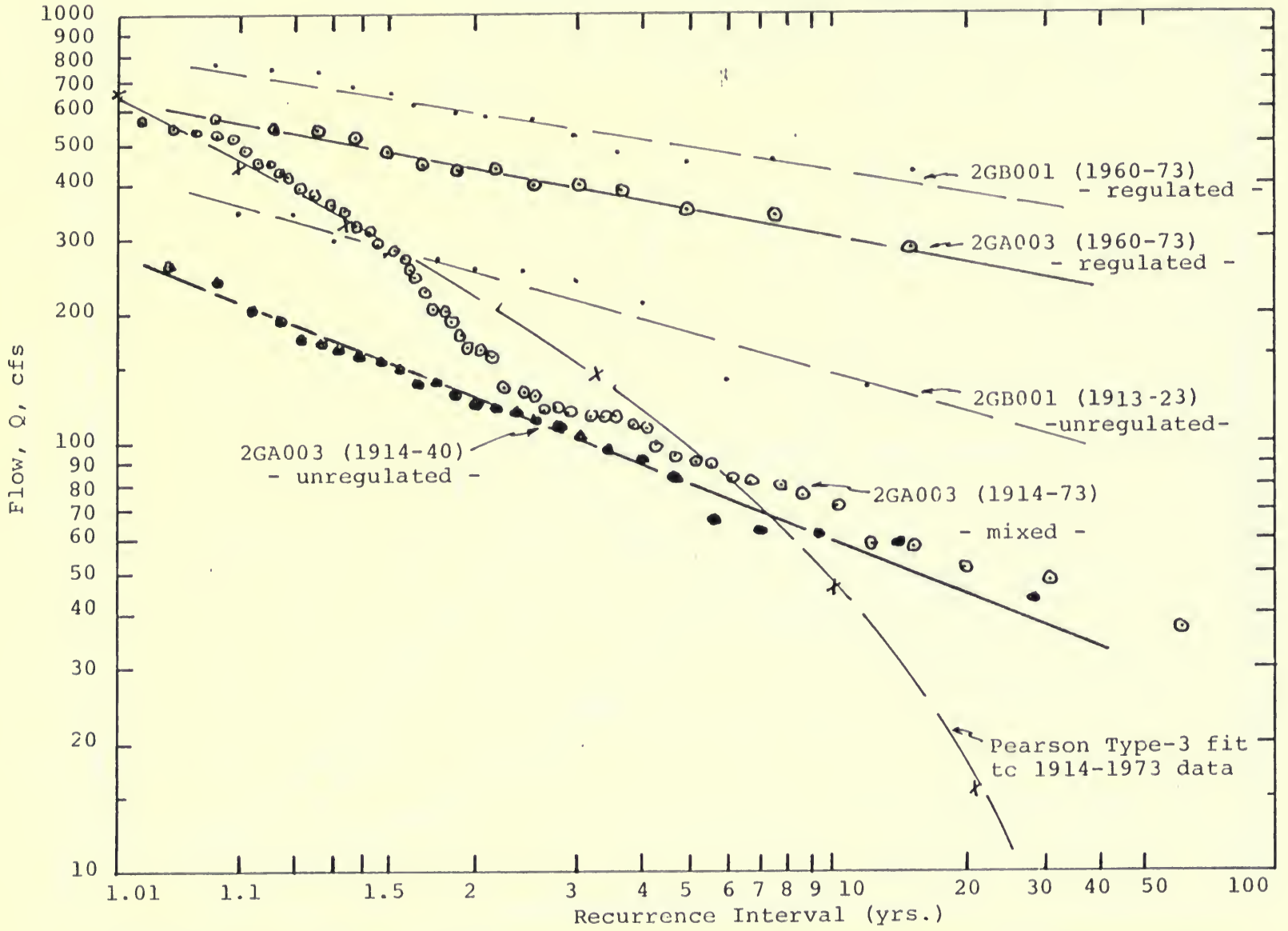

FIGURE 8.1: Frequency Curves for 7-day Average Minimum Flows: Effect of Flow Regulations - Stations Grand River at Galt (2GA003) and at Brantford (2GB001)

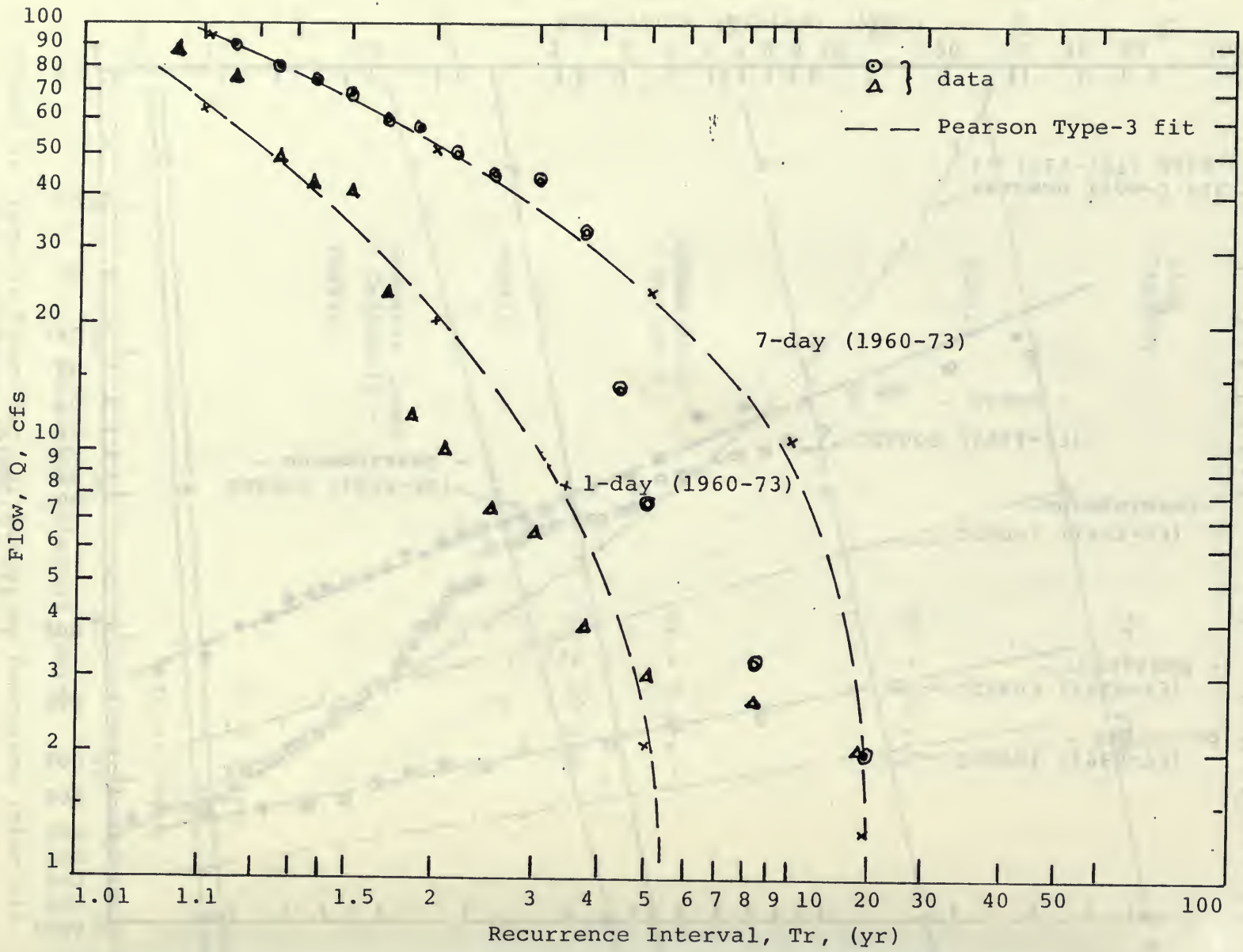

FIGURE 8.2: Frequency Curve for 1- and 7-day Average Minimum Flow: Effect of Regulated Flow 2GA028 - Conestogo at Glen Allan

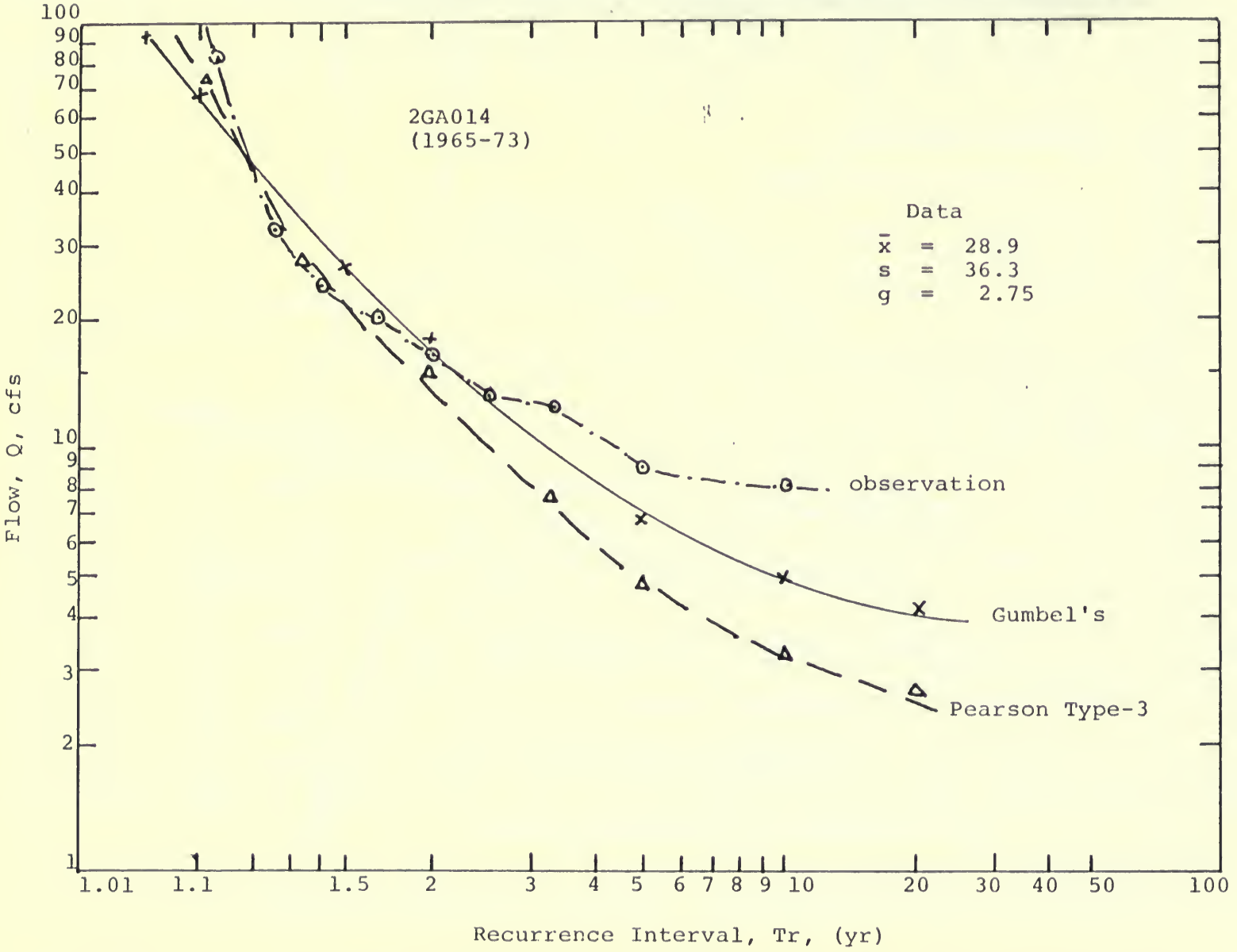

FIGURE 9 : Example of Poor Fit of Gumbel's and Pearson Type-3 Distributions to Low Flow Sequence (60-day average)

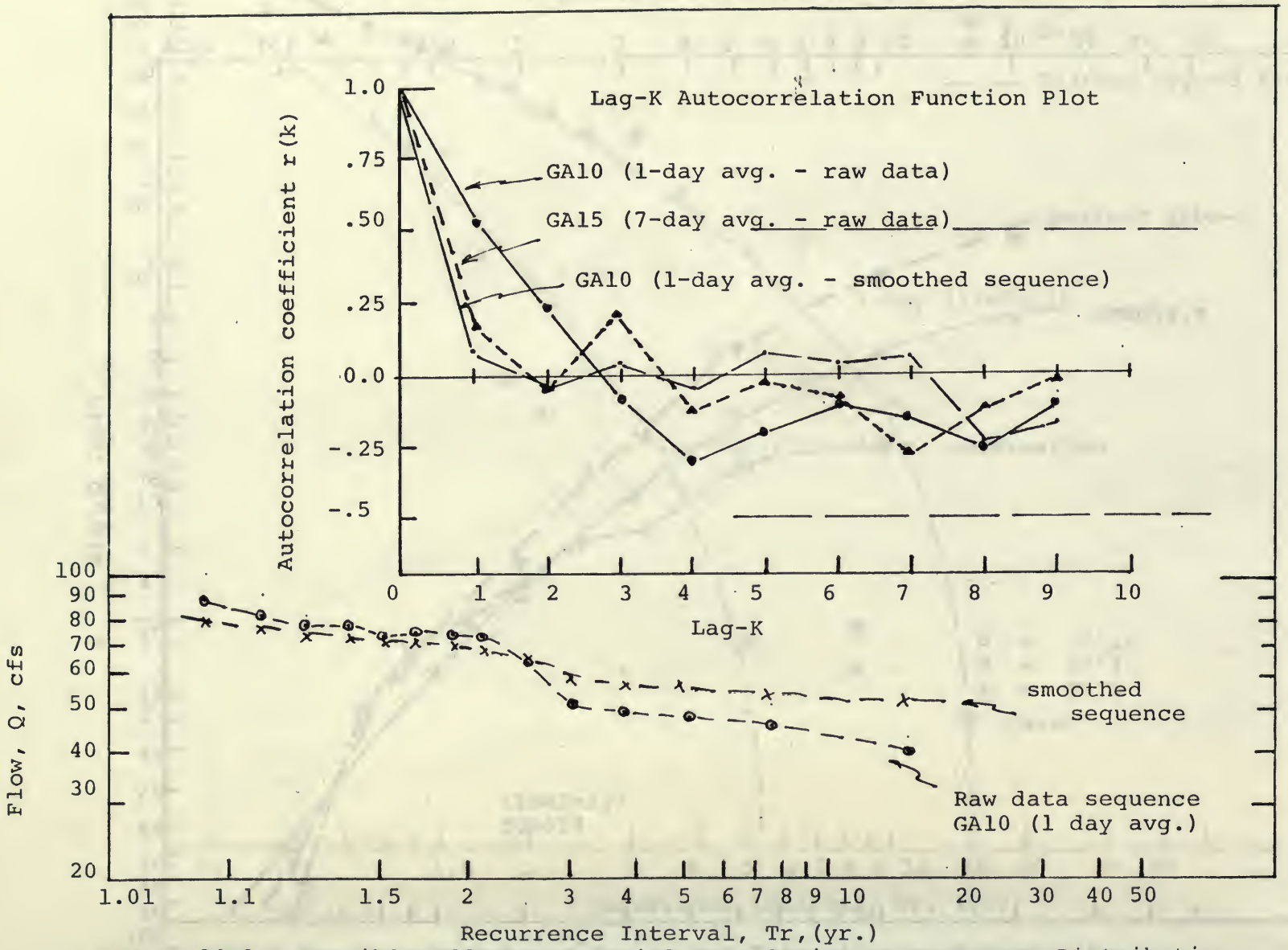
Lag-K Autocorrelation Function Plot
Autocorrelation coefficient r(k)
1.0
.75
.50
.25
0.0
-.25
-.5
GA10 (1-day avg. - raw data)
GA15 (7-day avg. - raw data)
GA10 (1-day avg. - smoothed sequence)
0 1 2 3 4 5 6 7 8 9 10
Lag-K
Flow, Q, cfs
100 90 80 70 60 50 40 30 20
smoothed sequence
Raw data sequence GA10 (1 day avg.)
1.01 1.1 1.5 2 3 4 5 6 7 8 9 10 20 30 40 50
Recurrence Interval, Tr,(yr.)

FIGURE 10.1: Possible Effects of Serial Correlation on Frequency Distribution of Low Flow Sequence

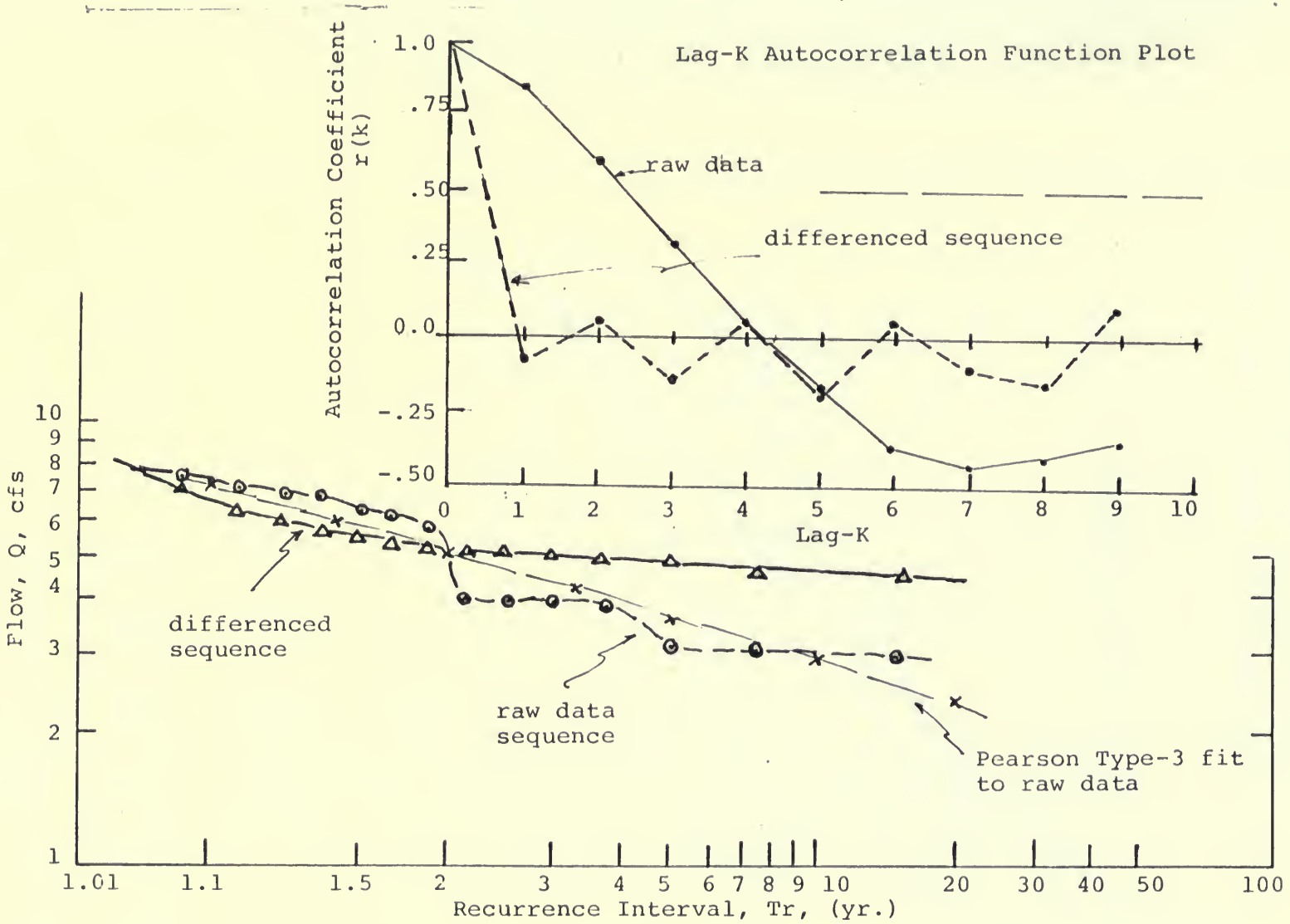

FIGURE 10.2: Possible Effects of Seasonality on Frequency Distribution of Low Flow Sequence - 2GA023. Canagagigue Creek near Elmira

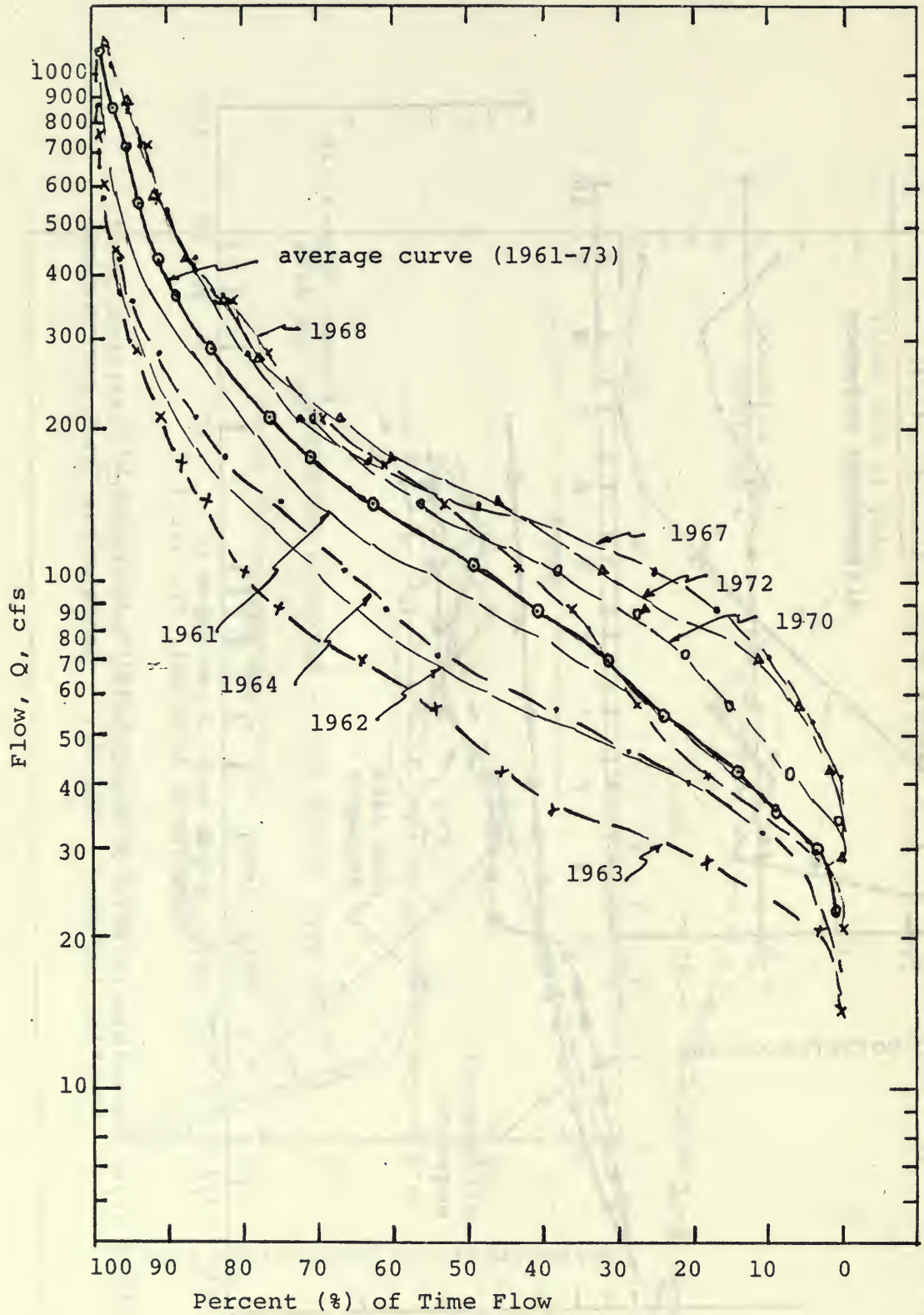

FIGURE 11: Family of Flow Duration Curves:
Speed River at Guelph (2GA015)

TABLE 1.1

LOW FLOW STATISTICS - 7-DAY AVERAGE

MINIMUM FLOW: 2GA014, 2GA015 and 2GA010

	2GA015	2GA014	2GA010
n	20	8	14
$\bar{x}$	34.9 cfs	6.7 cfs	69.8 cfs
S	11.1 cfs	3.2 cfs	14.84 cfs
g_1	0.35	0.1	-0.36
M_3	4.09×10^3	2.29	-9.31×10^2
μ	38.21 cfs	1.09 cfs	75.73 cfs
ε	8.91 cfs	-9.3 cfs	-5.19 cfs

TABLE 1.2

DESIGN FLOW $7Q_{20}$ FOR DIFFERENT PROBABILITY DISTRIBUTION MODELS

Station	$7Q_{2o}$ (cfs) DF	N	LN	G	P
2GA010	46.0	45.0	48.0	45.0	45.0
2GA014	1.0	1.1	2.6	-	1.5
2GA015	15.0	11.0	19.5	18.0	20.0

DF - distribution-free

N - normal

LN - log-normal

G - Gumbel's

P - Pearson Type-3

2GA010 - Nith River near Canning
2GA014 - Grand River near Marsville
2GA015 - Speed River at Guelph

8. APPENDIX

8. APPENDIX

Table 1a) Computation for Speed River at Guelph
7-Day Average Low Flow - Gumbel's Extremal Distribution

$\bar{x}$ = 34.9 = 0.3978 $\mu = \bar{x} + A\,S = 34.9 + 3.31446 = 38.214$
S = 11.1 A = 0.2986 $\varepsilon = \mu - B\,S = 38.214 - 29.302 = 8.912$
M^3 = .41 B = 2.6398 $\log(\mu-\varepsilon) = \log(38.214 - 8.912) = 1.467$
g_1 = .35 n = 20

Probability		$\log(x-\varepsilon) = \log(\mu-\varepsilon) +$	log y			
T	P	log y	log y	log(x-ε)	(x-ε)	x
21.0	.048	-1.2918	-0.5139	0.9531	8.9763	17.888
10.5	.095	-1.0008	-0.3981	1.0689	11.719	20.631
7.0	.143	-0.8115	-0.3228	1.1442	13.938	22.850
5.2	.190	-0.6760	-0.2689	1.1981	15.780	24.692
4.2	.238	-0.5657	-0.2250	1.2420	17.458	26.37
3.5	.286	-0.4725	-0.1880	1.2790	19.011	27.923
3.0	.333	-0.3926	-0.1562	1.3108	20.455	29.367
2.62	.381	-0.3191	-0.1269	1.3401	21.883	30.795
2.33	.429	-0.2515	-0.1000	1.3670	23.281	32.193
2.10	.476	-0.1896	-0.0754	1.3916	24.638	33.550
1.91	.524	-0.1294	-0.0515	1.4155	26.032	34.944
1.75	.571	-0.0725	-0.0288	1.4382	27.428	36.340
1.62	.619	-0.0155	-0.0062	1.4608	28.893	37.805
1.50	.667	0.0412	0.0164	1.4834	30.437	39.349
1.40	.714	0.0975	0.0388	1.5058	32.048	40.960
1.31	.762	0.1570	0.0624	1.5294	33.838	42.75
1.24	.810	0.2203	0.0876	1.5546	35.859	44.771
1.167	.857	0.2889	0.1149	1.5819	38.186	47.098
1.105	.905	0.3718	0.1479	1.6149	41.200	50.112
1.05	.952	0.4824	0.1919	1.6589	45.593	54.505

Table 1b) Computation for Speed River at Guelph
7-Day Average Low Flow - Pearson Type-3
Distribution

$\bar{x} = 34.9$
$S = 11.1$
$g_1 = 0.968$
$M_3 = 0.41$

$V = \frac{8}{g_1^2} = 8.5377$

$C = \frac{2}{g_1} = 2.0661$

$2C = 4.2688$

$x = \bar{x} + ks$

$k = \frac{x^2(P_1v)}{2c} - c$

T	P	$x^2(P_1v)$	$x^2(P_1v)/2c$	k	Sk	x
21.0	.048	3.06	0.7405	-1.3256	-14.714	20.186
10.5	.095	3.73	0.9023	-1.1638	-12.918	21.982
7.0	.143	4.10	0.9922	-1.0738	-11.919	22.981
5.2	.190	4.65	1.1253	-0.9408	-10.443	24.457
4.2	.238	5.32	1.2874	-0.7787	- 8.644	26.256
3.5	.286	5.91	1.4302	-0.6359	- 7.0585	27.84
3.0	.333	6.20	1.5004	-0.5657	- 6.2793	28.62
2.62	.381	6.58	1.5923	-0.4738	- 5.2592	29.64
2.33	.429	7.00	1.6940	-0.3721	- 4.1303	30.77
2.10	.476	7.58	1.8344	-0.2317	- 2.5719	32.328
1.91	.524	7.98	1.9312	-0.1349	- 1.4974	33.40
1.75	.571	8.10	1.9602	-0.1059	- 1.1755	33.72
1.62	.619	8.95	2.1659	0.0998	1.1078	36.01
1.50	.667	9.34	2.2603	0.1942	2.1556	37.06
1.40	.714	9.96	2.4103	0.3442	3.8206	38.72
1.31	.762	10.98	2.6571	0.5910	6.5601	41.4
1.24	.810	11.48	2.7782	0.7121	7.9043	42.80
1.167	.857	13.00	3.1460	1.0799	11.987	46.89
1.105	.905	14.12	3.4171	1.3510	14.996	49.89
1.05	.952	16.29	3.9422	1.8761	20.825	55.72

TABLE 2a: Values of Frequency Factors: $1/\alpha$, $A\alpha$ and $B\alpha$ for Given Skew Coefficients in Gumbel's Distribution

Coefficient of Skew	Scale Parameter	Frequency Factors		Coefficient of Skew	Scale Parameter	Frequency Factors	
g_1	$1/\alpha$	A_α	B_α	g_1	$1/\alpha$	A_α	B_α
				1.054	0.66	0.162	1.649
				1.081	0.67	0.157	1.623
0.007	0.28	0.355	3.573	1.107	0.68	0.152	1.598
0.038	0.29	0.350	3.468	1.134	0.69	0.147	1.573
0.069	0.30	0.346	3.370	1.160	0.70	0.142	1.549
0.099	0.31	0.341	3.277	1.187	0.71	0.136	1.526
0.129	0.32	0.336	3.190	1.214	0.72	0.131	1.503
0.158	0.33	0.331	3.108	1.240	0.73	0.126	1.480
0.188	0.34	0.327	3.030	1.267	0.74	0.121	1.458
0.217	0.35	0.322	2.955	1.294	0.75	0.116	1.436
0.245	0.36	0.317	2.885	1.321	0.76	0.111	1.415
0.274	0.37	0.312	2.818	1.348	0.77	0.106	1.394
0.302	0.38	0.307	2.754	1.375	0.78	0.101	1.374
0.331	0.39	0.302	2.692	1.402	0.79	0.096	1.354
0.359	0.40	0.297	2.634	1.430	0.80	0.092	1.334
0.386	0.41	0.292	2.578	1.457	0.81	0.087	1.314
0.414	0.42	0.287	2.524	1.484	0.82	0.082	1.295
0.442	0.43	0.282	2.472	1.512	0.83	0.077	1.276
0.469	0.44	0.277	2.422	1.540	0.84	0.072	1.258
0.496	0.45	0.271	2.374	1.567	0.85	0.067	1.240
0.523	0.46	0.266	2.328	1.595	0.86	0.063	1.222
0.551	0.47	0.261	2.284	1.623	0.87	0.058	1.204
0.577	0.48	0.256	2.241	1.651	0.88	0.053	1.187
0.604	0.49	0.251	2.199	1.680	0.89	0.049	1.170
0.631	0.50	0.246	2.159	1.708	0.90	0.044	1.154
0.658	0.51	0.240	2.120	1.737	0.91	0.040	1.137
0.684	0.52	0.235	2.082	1.765	0.92	0.035	1.121
0.711	0.53	0.230	2.045	1.794	0.93	0.031	1.105
0.738	0.54	0.225	2.009	1.823	0.94	0.026	1.089
0.764	0.55	0.219	1.975	1.852	0.95	0.022	1.074
0.790	0.56	0.214	1.941	1.881	0.96	0.017	1.059
0.817	0.57	0.209	1.909	1.911	0.97	0.013	1.044
0.843	0.58	0.204	1.877	1.940	0.98	0.009	1.029
0.870	0.59	0.199	1.846	1.970	0.99	0.004	1.014
0.896	0.60	0.193	1.815	2.000	1.00	0.0	1.000
0.922	0.61	0.188	1.786	2.309	1.1	-0.040	0.867
0.949	0.62	0.183	1.757	2.640	1.2	-0.077	0.752
0.975	0.63	0.178	1.729	2.996	1.3	-0.109	0.652
1.002	0.64	0.172	1.702	3.382	1.4	-0.136	0.563
1.028	0.65	0.167	1.675	3.802	1.5	-0.160	0.486

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TABLE 2a: (continued)

Coefficient of Skew	Scale Parameter	Frequency Factors		Coefficient of Skew	Scale Parameter	Frequency Factors	
g_1	$1/\alpha$	A_α	B_α	g_1	$1/\alpha$	A_α	B_α
-1.081	0.01	0.448	78.982	-0.398	0.16	0.405	5.765
-1.025	0.02	0.446	39.989	-0.361	0.17	0.401	5.368
-0.971	0.03	0.444	26.986	-0.325	0.18	0.397	5.205
-0.918	0.04	0.442	20.481	-0.289	0.19	0.393	4.969
-0.868	0.05	0.439	16.574	-0.254	0.20	0.389	4.755
-0.819	0.06	0.437	13.967	-0.220	0.21	0.385	4.561
-0.772	0.07	0.434	12.103	-0.186	0.22	0.381	4.383
-0.726	0.08	0.431	10.702	-0.152	0.23	0.377	4.221
-0.681	0.09	0.428	9.611	-0.120	0.24	0.372	4.071
-0.638	0.10	0.425	8.737	-0.087	0.25	0.368	3.933
-0.595	0.11	0.422	8.020	-0.055	0.26	0.364	3.804
-0.554	0.12	0.419	7.421	-0.024	0.27	0.359	3.684
-0.514	0.13	0.415	6.913				
-0.474	0.14	0.412	6.476				
-0.436	0.15	0.408	6.097				

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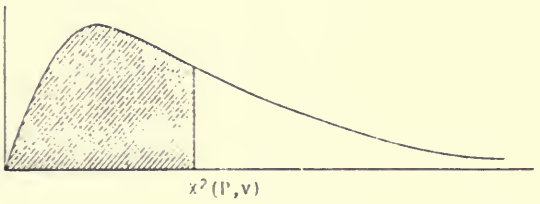
TABLE 2b: Values of Frequency Factor k for Given Skew Coefficient and Probability Levels in Pearson Type-3 Distribution

g_1	Probability																
	.99	.97	.95	.90	.80	.75	.70	.60	.50	.40	.30	.25	.20	.10	.05	.03	.01
0.00	2.33	1.88	1.64	1.28	0.84	0.67	0.52	0.25	0.00	-0.25	-0.52	-0.67	-0.84	-1.28	-1.64	-1.88	-2.33
0.05	2.36	1.90	1.65	1.28	0.84	0.66	0.52	0.24	-0.01	-0.26	-0.52	-0.68	-0.84	-1.28	-1.62	-1.86	-2.29
0.10	2.40	1.92	1.67	1.29	0.84	0.66	0.51	0.24	-0.02	-0.27	-0.53	-0.68	-0.85	-1.27	-1.61	-1.84	-2.25
0.15	2.44	1.94	1.68	1.30	0.84	0.66	0.50	0.23	-0.02	-0.28	-0.54	-0.68	-0.85	-1.26	-1.60	-1.82	-2.22
0.20	2.47	1.96	1.70	1.30	0.83	0.65	0.50	0.22	-0.03	-0.28	-0.55	-0.69	-0.85	-1.26	-1.58	-1.79	-2.18
0.25	2.50	1.98	1.71	1.30	0.82	0.64	0.49	0.21	-0.04	-0.29	-0.56	-0.70	-0.85	-1.25	-1.56	-1.77	-2.14
0.30	2.54	2.00	1.72	1.31	0.82	0.64	0.48	0.20	-0.05	-0.30	-0.56	-0.70	-0.85	-1.24	-1.55	-1.75	-2.10
0.35	2.58	2.02	1.73	1.32	0.82	0.64	0.48	0.20	-0.06	-0.30	-0.56	-0.70	-0.85	-1.24	-1.53	-1.72	-2.06
0.40	2.61	2.04	1.75	1.32	0.82	0.63	0.47	0.19	-0.07	-0.31	-0.57	-0.71	-0.85	-1.23	-1.52	-1.70	-2.03
0.45	2.64	2.06	1.76	1.32	0.82	0.62	0.46	0.18	-0.08	-0.32	-0.58	-0.71	-0.85	-1.22	-1.51	-1.68	-2.00
0.50	2.68	2.08	1.77	1.32	0.81	0.62	0.46	0.17	-0.08	-0.33	-0.58	-0.71	-0.85	-1.22	-1.49	-1.66	-1.96
0.55	2.72	2.10	1.78	1.32	0.80	0.62	0.45	0.16	-0.09	-0.34	-0.58	-0.72	-0.85	-1.21	-1.47	-1.64	-1.92
0.60	2.75	2.12	1.80	1.33	0.80	0.61	0.44	0.16	-0.10	-0.34	-0.59	-0.72	-0.85	-1.20	-1.45	-1.61	-1.88
0.65	2.78	2.14	1.81	1.33	0.80	0.60	0.44	0.15	-0.11	-0.35	-0.60	-0.72	-0.85	-1.19	-1.44	-1.59	-1.84
0.70	2.82	2.15	1.82	1.33	0.79	0.59	0.43	0.14	-0.12	-0.36	-0.60	-0.72	-0.85	-1.18	-1.42	-1.57	-1.81
0.75	2.86	2.16	1.83	1.34	0.78	0.58	0.42	0.13	-0.12	-0.36	-0.60	-0.72	-0.86	-1.18	-1.40	-1.54	-1.78
0.80	2.89	2.18	1.84	1.34	0.78	0.58	0.41	0.12	-0.13	-0.37	-0.60	-0.73	-0.86	-1.17	-1.38	-1.52	-1.74
0.85	2.92	2.20	1.85	1.34	0.78	0.58	0.40	0.12	-0.14	-0.38	-0.60	-0.73	-0.86	-1.16	-1.36	-1.49	-1.70
0.90	2.96	2.22	1.86	1.34	0.77	0.57	0.40	0.11	-0.15	-0.38	-0.61	-0.73	-0.85	-1.15	-1.35	-1.47	-1.66
0.95	2.99	2.24	1.87	1.34	0.76	0.56	0.39	0.10	-0.16	-0.38	-0.62	-0.73	-0.85	-1.14	-1.34	-1.44	-1.62
1.00	3.02	2.25	1.88	1.34	0.76	0.55	0.38	0.09	-0.16	-0.39	-0.62	-0.73	-0.85	-1.13	-1.32	-1.42	-1.59
1.05	3.06	2.26	1.88	1.34	0.75	0.54	0.37	0.08	-0.17	-0.40	-0.62	-0.74	-0.85	-1.12	-1.30	-1.40	-1.56
1.10	3.09	2.28	1.89	1.34	0.74	0.54	0.36	0.07	-0.17	-0.41	-0.62	-0.74	-0.85	-1.10	-1.28	-1.38	-1.52
1.15	3.12	2.30	1.90	1.34	0.74	0.53	0.36	0.06	-0.18	-0.42	-0.62	-0.74	-0.84	-1.09	-1.26	-1.36	-1.48
1.20	3.15	2.31	1.91	1.34	0.73	0.52	0.35	0.05	-0.19	-0.42	-0.63	-0.74	-0.84	-1.08	-1.24	-1.33	-1.45
1.25	3.18	2.32	1.92	1.34	0.72	0.52	0.34	0.04	-0.20	-0.42	-0.63	-0.74	-0.84	-1.07	-1.22	-1.30	-1.42
1.30	3.21	2.34	1.92	1.34	0.72	0.51	0.33	0.04	-0.21	-0.43	-0.63	-0.74	-0.84	-1.06	-1.20	-1.28	-1.38
1.35	3.24	2.36	1.93	1.34	0.72	0.50	0.32	0.03	-0.22	-0.44	-0.64	-0.74	-0.84	-1.05	-1.18	-1.26	-1.35
1.40	3.27	2.37	1.94	1.34	0.71	0.49	0.31	0.02	-0.22	-0.44	-0.64	-0.75	-0.83	-1.04	-1.17	-1.23	-1.32
1.45	3.30	2.38	1.94	1.34	0.70	0.48	0.30	0.01	-0.23	-0.44	-0.64	-0.75	-0.82	-1.03	-1.15	-1.21	-1.29
1.50	3.33	2.39	1.95	1.33	0.69	0.47	0.30	0.00	-0.24	-0.45	-0.61	-0.75	-0.82	-1.02	-1.13	-1.19	-1.26
1.55	3.36	2.40	1.96	1.33	0.69	0.46	0.29	-0.01	-0.24	-0.46	-0.64	-0.75	-0.82	-1.00	-1.12	-1.16	-1.23
1.60	3.39	2.42	1.96	1.33	0.68	0.46	0.28	-0.02	-0.25	-0.46	-0.64	-0.75	-0.81	-0.99	-1.10	-1.14	-1.20
1.65	3.42	2.43	1.96	1.32	0.67	0.45	0.27	-0.02	-0.26	-0.46	-0.64	-0.74	-0.81	-0.98	-1.08	-1.12	-1.17
1.70	3.44	2.44	1.97	1.32	0.66	0.44	0.26	-0.03	-0.27	-0.47	-0.64	-0.74	-0.81	-0.97	-1.06	-1.10	-1.11
1.75	3.47	2.45	1.98	1.32	0.65	0.43	0.25	-0.04	-0.28	-0.48	-0.61	-0.72	-0.80	-0.96	-1.04	-1.08	-1.12
1.80	3.50	2.46	1.98	1.32	0.64	0.42	0.24	-0.05	-0.28	-0.48	-0.64	-0.72	-0.80	-0.94	-1.02	-1.06	-1.09
1.85	3.52	2.48	1.98	1.32	0.64	0.41	0.23	-0.06	-0.28	-0.48	-0.64	-0.72	-0.80	-0.93	-1.00	-1.04	-1.06
1.90	3.55	2.49	1.99	1.31	0.63	0.40	0.22	-0.07	-0.29	-0.48	-0.64	-0.72	-0.79	-0.92	-0.98	-1.01	-1.04
1.95	3.58	2.50	2.00	1.30	0.62	0.40	0.21	-0.08	-0.30	-0.48	-0.64	-0.72	-0.78	-0.91	-0.96	-0.99	-1.02
2.00	3.60	2.51	2.00	1.30	0.61	0.39	0.20	-0.08	-0.31	-0.49	-0.64	-0.71	-0.78	-0.90	-0.95	-0.97	-0.99
2.05	3.63	2.52	2.00	1.30	0.60	0.39	0.20	-0.09	-0.32	-0.49	-0.64	-0.71	-0.77	-0.89	-0.94	-0.95	-0.96
2.10	3.65	2.53	2.00	1.29	0.60	0.38	0.19	-0.10	-0.32	-0.49	-0.64	-0.70	-0.76	-0.88	-0.93	-0.93	-0.94
2.15	3.68	2.54	2.01	1.28	0.59	0.38	0.18	-0.10	-0.32	-0.49	-0.63	-0.70	-0.76	-0.86	-0.92	-0.92	-0.92
2.20	3.70	2.55	2.01	1.28	0.58	0.37	0.17	-0.11	-0.33	-0.49	-0.63	-0.69	-0.75	-0.85	-0.90	-0.90	-0.90
2.25	3.72	2.56	2.01	1.27	0.57	0.36	0.16	-0.12	-0.34	-0.49	-0.63	-0.68	-0.74	-0.83	-0.88	-0.88	-0.89
2.30	3.75	2.56	2.01	1.27	0.56	0.35	0.15	-0.12	-0.34	-0.49	-0.62	-0.68	-0.73	-0.82	-0.86	-0.86	-0.87
2.35	3.77	2.56	2.01	1.26	0.55	0.34	0.14	-0.13	-0.34	-0.50	-0.62	-0.67	-0.72	-0.81	-0.84	-0.81	-0.85
2.40	3.79	2.57	2.01	1.25	0.54	0.33	0.13	-0.14	-0.35	-0.50	-0.62	-0.66	-0.71	-0.79	-0.82	-0.82	-0.83
2.45	3.81	2.58	2.01	1.25	0.54	0.32	0.13	-0.14	-0.35	-0.50	-0.62	-0.66	-0.70	-0.78	-0.80	-0.80	-0.82
2.50	3.83	2.58	2.01	1.24	0.53	0.32	0.12	-0.15	-0.36	-0.50	-0.61	-0.65	-0.70	-0.77	-0.79	-0.79	-0.80
2.55	3.85	2.58	2.01	1.23	0.52	0.31	0.11	-0.16	-0.36	-0.50	-0.61	-0.65	-0.69	-0.75	-0.78	-0.78	-0.78
2.60	3.87	2.59	2.01	1.23	0.51	0.30	0.10	-0.17	-0.37	-0.50	-0.60	-0.64	-0.68	-0.74	-0.76	-0.76	-0.77
2.65	3.89	2.59	2.01	1.22	0.50	0.29	0.09	-0.18	-0.37	-0.50	-0.60	-0.64	-0.67	-0.73	-0.75	-0.75	-0.75
2.70	3.91	2.60	2.01	1.21	0.49	0.28	0.08	-0.18	-0.38	-0.50	-0.60	-0.63	-0.67	-0.72	-0.73	-0.73	-0.74
2.75	3.93	2.61	2.02	1.21	0.48	0.27	0.07	-0.19	-0.38	-0.50	-0.59	-0.63	-0.66	-0.71	-0.72	-0.72	-0.72
2.80	3.95	2.61	2.02	1.20	0.47	0.27	0.06	-0.20	-0.38	-0.50	-0.59	-0.62	-0.65	-0.70	-0.71	-0.71	-0.71
2.85	3.97	2.62	2.02	1.20	0.46	0.26	0.05	-0.21	-0.39	-0.50	-0.59	-0.62	-0.64	-0.69	-0.70	-0.70	-0.70
2.90	3.99	2.62	2.02	1.19	0.45	0.26	0.04	-0.21	-0.39	-0.50	-0.58	-0.61	-0.64	-0.67	-0.68	-0.68	-0.69
2.95	4.00	2.62	2.02	1.18	0.44	0.25	0.01	-0.22	-0.40	-0.50	-0.58	-0.61	-0.63	-0.66	-0.67	-0.67	-0.68
3.00	4.02	2.63	2.02	1.18	0.42	0.25	0.03	-0.23	-0.40	-0.50	-0.57	-0.60	-0.62	-0.65	-0.66	-0.66	-0.67

TABLE 3: Percentile Values for Chi-square, χ^2, Distribution

PERCENTILE VALUES [$\chi^2(P,\nu)$]
for
THE CHI-SQUARE DISTRIBUTION
with ν degrees of freedom
(shaded area = P)



ν	$\chi^2_{.995}$	$\chi^2_{.99}$	$\chi^2_{.975}$	$\chi^2_{.95}$	$\chi^2_{.90}$	$\chi^2_{.75}$	$\chi^2_{.50}$	$\chi^2_{.25}$	$\chi^2_{.10}$	$\chi^2_{.05}$	$\chi^2_{.025}$	$\chi^2_{.01}$	$\chi^2_{.005}$
1	7.88	6.63	5.02	3.84	2.71	1.32	.455	.102	.0158	.0039	.0010	.0002	.0000
2	10.6	9.21	7.38	5.99	4.61	2.77	1.39	.575	.211	.103	.0506	.0201	.0100
3	12.8	11.3	9.35	7.81	6.25	4.11	2.37	1.21	.584	.352	.216	.115	.072
4	14.9	13.3	11.1	9.49	7.78	5.39	3.36	1.92	1.06	.711	.484	.297	.207
5	16.7	15.1	12.8	11.1	9.24	6.63	4.35	2.67	1.61	1.15	.831	.554	.412
6	18.5	16.8	14.4	12.6	10.6	7.84	5.35	3.45	2.20	1.64	1.24	.872	.676
7	20.3	18.5	16.0	14.1	12.0	9.04	6.35	4.25	2.83	2.17	1.69	1.24	.989
8	22.0	20.1	17.5	15.5	13.4	10.2	7.34	5.07	3.49	2.73	2.18	1.65	1.34
9	23.6	21.7	19.0	16.9	14.7	11.4	8.34	5.90	4.17	3.33	2.70	2.09	1.73
10	25.2	23.2	20.5	18.3	16.0	12.5	9.34	6.74	4.87	3.94	3.25	2.56	2.16
11	26.8	24.7	21.9	19.7	17.3	13.7	10.3	7.58	5.58	4.57	3.82	3.05	2.60
12	28.3	26.2	23.3	21.0	18.5	14.8	11.3	8.44	6.30	5.23	4.40	3.57	3.07
13	29.8	27.7	24.7	22.4	19.8	16.0	12.3	9.30	7.04	5.89	5.01	4.11	3.57
14	31.3	29.1	26.1	23.7	21.1	17.1	13.3	10.2	7.79	6.57	5.63	4.66	4.07
15	32.8	30.6	27.5	25.0	22.3	18.2	14.3	11.0	8.55	7.26	6.26	5.23	4.60
16	34.3	32.0	28.8	26.3	23.5	19.4	15.3	11.9	9.31	7.96	6.91	5.81	5.14
17	35.7	33.4	30.2	27.6	24.8	20.5	16.3	12.8	10.1	8.67	7.56	6.41	5.70
18	37.2	34.8	31.5	28.9	26.0	21.6	17.3	13.7	10.9	9.39	8.23	7.01	6.26
19	38.6	36.2	32.9	30.1	27.2	22.7	18.3	14.6	11.7	10.1	8.91	7.63	6.84
20	40.0	37.6	34.2	31.4	28.4	23.8	19.3	15.5	12.4	10.9	9.59	8.26	7.43
21	41.4	38.9	35.5	32.7	29.6	24.9	20.3	16.3	13.2	11.6	10.3	8.90	8.03
22	42.8	40.3	36.8	33.9	30.8	26.0	21.3	17.2	14.0	12.3	11.0	9.54	8.64
23	44.2	41.6	38.1	35.2	32.0	27.1	22.3	18.1	14.8	13.1	11.7	10.2	9.26
24	45.6	43.0	39.4	36.4	33.2	28.2	23.3	19.0	15.7	13.8	12.4	10.9	9.89
25	46.9	44.3	40.6	37.7	34.4	29.3	24.3	19.9	16.5	14.6	13.1	11.5	10.5
26	48.3	45.6	41.9	38.9	35.6	30.4	25.3	20.8	17.3	15.4	13.8	12.2	11.2
27	49.6	47.0	43.2	40.1	36.7	31.5	26.3	21.7	18.1	16.2	14.6	12.9	11.8
28	51.0	48.3	44.5	41.3	37.9	32.6	27.3	22.7	18.9	16.9	15.3	13.6	12.5
29	52.3	49.6	45.7	42.6	39.1	33.7	28.3	23.6	19.8	17.7	16.0	14.3	13.1
30	53.7	50.9	47.0	43.8	40.3	34.8	29.3	24.5	20.6	18.5	16.8	15.0	13.8
40	66.8	63.7	59.3	55.8	51.8	45.6	39.3	33.7	29.1	26.5	24.1	22.2	20.7
50	79.5	76.2	71.4	67.5	63.2	56.3	49.3	42.9	37.7	34.8	32.1	29.7	28.0
60	92.0	88.4	83.3	79.1	74.4	67.0	59.3	52.3	46.5	43.2	40.5	37.5	35.5
70	104.2	100.4	95.0	90.5	85.5	77.6	69.3	61.7	55.3	51.7	48.8	45.4	43.3
80	116.3	112.3	106.6	101.9	96.6	88.1	79.3	71.1	64.3	60.4	57.2	53.5	51.2
90	128.3	124.1	118.1	113.1	107.6	98.6	89.3	80.6	73.3	69.1	65.6	61.8	59.2
100	140.2	135.8	129.6	124.3	118.5	109.1	99.3	90.1	82.4	77.9	74.2	70.1	67.3

APPENDIX II

STREAM ASSIMILATIVE CAPACITY - A GRAPHICAL PRODECURE

F.N. Hamilton

STREAM ASSIMILATIVE CAPACITY - A GRAPHICAL PROCEDURE

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April, 1976

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PREFACE

The methodology presented in this manual is a simplified graphical approach to waste assimilation analysis. The analysis is structured so that water quality responses can be estimated with a minimum amount of information. This does not preclude the use of any available water quality and quantity data, available data should be scrutinized for use in the analysis.

Discretion is to be used when interpreting the results of the analysis especially with respect to treatment levels and projected water quality effects, they should be regarded only as engineering estimates and not as precise solutions. Much of the material presented in this manual was taken from "Simplified Mathematical Modelling of Water Quality", prepared by Hydroscience Inc. for the EPA, March 1971. It was revised and adapted to reflect the criteria and conditions for Ontario.

INTRODUCTION

Mathematical models are being used extensively to analyse water quality in rivers and streams, estuaries and impoundments. This report outlines the application of the dissolved oxygen model in a simplified manner to rivers and streams. In relating wastewater discharges to water quality in the receiving body, the water body is assumed to be one dimensional (i.e. longitudinal) and the basic geometry of the system is relatively simple. Only single point sources are considered, although distributed sources can be only approximated by a point source at its centre of gravity.

As with any type of waste loading assessment, some basic facts and knowledge of the subject are required. For example, it is important that the analyst has the resources available to calculate drought streamflows, measure or estimate upstream water quality conditons, determine maximum water temperatures, etc.

The method presented in this manual has definite advantages over other "arm-chair" assessment methods traditionally employed. It also has certain limitations that the analyst must take into consideration.

Advantages:

1. In-depth knowledge of dissolved oxygen balance equaitons, reaction rate coefficients, etc. is not required.

2. A computer or sophisticated desk-top calculator is not required.

3. With the appropriate input material at hand, calculations can be completed in a few minutes.

4. A wide variety of options can be quickly and easily tested.

Limitations:

1. The impact of aquatic floral photosynthesis and respiration, sludge demand, artificial aeration, etc. cannot be directly taken into account by this method.

2. A reasonably accurate knowledge of physical conditions of the stream (i.e. mean depth, velocity, bottom type, turbulence, etc.) is usually required and on occasions may necessitate a short reconnaissance field trip.

The report is presented in two sections together with an appendix. The first section outlines the technical data required for analysis, i.e. population, waste loads, treatment efficiencies, etc. Included are graphs and tables representing guidelines and ranges for the above mentioned parameters. The second section outlines the theory and development of the model together with worked examples to show how the nomograph is to be used.

The appendix discusses the reaction coefficients involved in the dissolved analysis in streams. The graphs included are based on actual field data and can be used where data is unavailable for the stream in question.

I TECHNICAL DATA REQUIRED FOR ANALYSIS

For a preliminary water quality analysis, of streams and rivers, the factors which determine the quantity of waste from treatment plant discharged to the stream, together with the geophysical characteristics of the stream are reviewed.

The quantity of waste depends on the size of the population, per capita waste quantities and the removal efficiency of the particular waste treatment process.

The geophysical features include the temperature, natural background water quality, structure and shape of the drainage system and the flow of the stream.

1. Waste Discharge

a. Population[1]- The design population Pd, is established by multiplying the present population Po, by the growth factor f_1, for that region:

$$Pd = f_1 Po \quad \ldots\ldots \quad 1.1$$

[1] *In most cases the design population is given. Present population and future population predictions figures can be obtained from the Ministry of Treasury Economics and Intergovernmental Affairs or the Regional Municipality concerned.*

b. Per Capita Waste Flow and Quality - varies from municipality to municipality depending on the characteristics of the region, e.g. population, the extent of industrialization and the hydrological feature of the region. Table 1 presents some guidelines for per capita waste flows. A typical municipal waste is 150 gallons per capita/day with an ultimate carbonaceous BOD and nitrogenous BOD of about 0.26 and 0.16 pounds per capita day respectively.

The waste flow and mass rate of discharge are therefore:

$$q = f_2 Pd = f_2 f_1 Po \ldots\ldots \quad 1.2$$

where:

q = wastewater flow (usually MIGD)

f_2 = per capita flow (gallons/capita-day)

and:

$$l = f_3 Pd = f_3 f_1 Po \ldots\ldots \quad 1.3$$

where:

l = mass loading rate to treatment plant (pounds/day)

f_3 = per capita contribution (pounds/capita-day)

TABLE 1

PER CAPITA WASTE FLOWS AND QUALITY

	f_2 gallons cap-day	Ultimate BOD lb/cap-day		Suspended Solids lb/cap-day	Nutrients lb/cap-day	
		BOD_c	BOD_n		Total N	Total P
Low	100	0.15	0.12	0.15	0.02	0.005
Average	150	0.26	0.16	0.30	0.04	0.01
High	225	0.40	0.23	0.45	0.05	0.018

TABLE II

ESTIMATED EFFICIENCY OF TREATMENT LEVELS

ULTIMATE OXYGEN DEMAND

Treatment Level	% Removal		#/Cap UOD Remaining		Total	f_4 Fraction UOD Remaining
	BOD_c	BOD_n	BOD_c	BOD_n		
1. Primary	60	10	0.104	0.144	0.248	0.59
2. Intermediate	80	15	0.052	0.136	0.188	0.45
3. Secondary	90	75	0.026	0.040	0.066	0.16
4. Advanced	95	95	0.013	0.008	0.021	0.05

c. Treatment Efficiences and Residuals

Treatment levels can be categorized into groups as follows:

1. Primary

Systems employing screening and sedimentation, includes private and communal septic tanks (range 40-60% BOD_5 removal).

2. Intermediate

Primary with phosphorus removal, some lagoon systems (range 60-80% BOD_5 removal).

3. Secondary

Systems employing treatment such as conventional activated sludge, trickling filters and waste stabilization ponds with phosphorus removal (range 80-90% BOD_5 removal).

4. Advanced

Systems employing treatment such as extended aeration (nitrification), contact stabilization, filtration and activated carbon (range 90-97% BOD_5 removal).

With these discrete treatment levels and the following assumptions, guidelines can be developed for overall treatment efficiency. Table II and III are based on the following assumptions.

a) 150 gallons/capita-day,

b) 0.26 pounds ultimate CBOD/capita-day,

c) 0.16 pounds ultimate NBOD/capita-day,

d) 0.01 pounds P/capita-day,

e) 0.04 pounds nitrogen/capita-day.

The efficiency guidelines in Tables II and III may be used to estimate the discharge waste load.

$$\text{i.e.: } W = f_4 I = f_4 f_3 f_1 Po \quad \ldots\ldots \quad 1.4$$

where: W = mass rate of waste material discharged to the receiving water (pounds/day)

f_4 = residual fraction after treatment (Tables II and III).

TABLE III

ESTIMATED EFFICIENCY OF TREATMENT LEVELS

PHOSPHORUS AND TOTAL NITROGEN

Treatment Level	Phosphorus Fraction Removed	Residual	#P/ Cap/day Remaining	TKN Fraction Converted	Residual	#TKN Cap/day Remaini
1. Primary	0.45	0.55	0.006	0.10	0.90	0.036
& P removal	0.80	0.20	0.002	0.10	0.90	0.036
2. Intermediate	0.80	0.20	0.002	0.15	0.85	0.034
3. Secondary	0.50	0.50	0.005	0.75	0.25	0.010
& P removal	0.80	0.20	0.002	0.75	0.25	0.010
4. Advance	$\geq$ 0.90	$\leq$ 0.10	$\leq$ 0.001	0.95	0.05	0.002

2. Drainage Basin Characteristics

a. Temperature - critical water quality conditions usually occur during the middle to late summer when the water temperature is high. The temperature affects the solubility of many substances as well as the rate coefficient of many reactions. In dissolved oxygen analysis, the saturation value of DO is a function of temperature, Figure 1.1 shows the variation of the saturation value with temperature.

b. Natural Background Quality - may vary depending on the characteristics of the land and its usage, and the rainfall and runoff patterns of the area. Such information may be available from historical data or river surveys, for dissolved oxygen deficit reasonable values for low, moderate and high are 0.5, 1.0 and 2.0 mg/l respectively.

The high value is indicative of highly organic and/or swampy areas while the lower value is associated with low organic mineral areas.

In the analysis discussed in the following section, any natural background water quality must be incorporated.

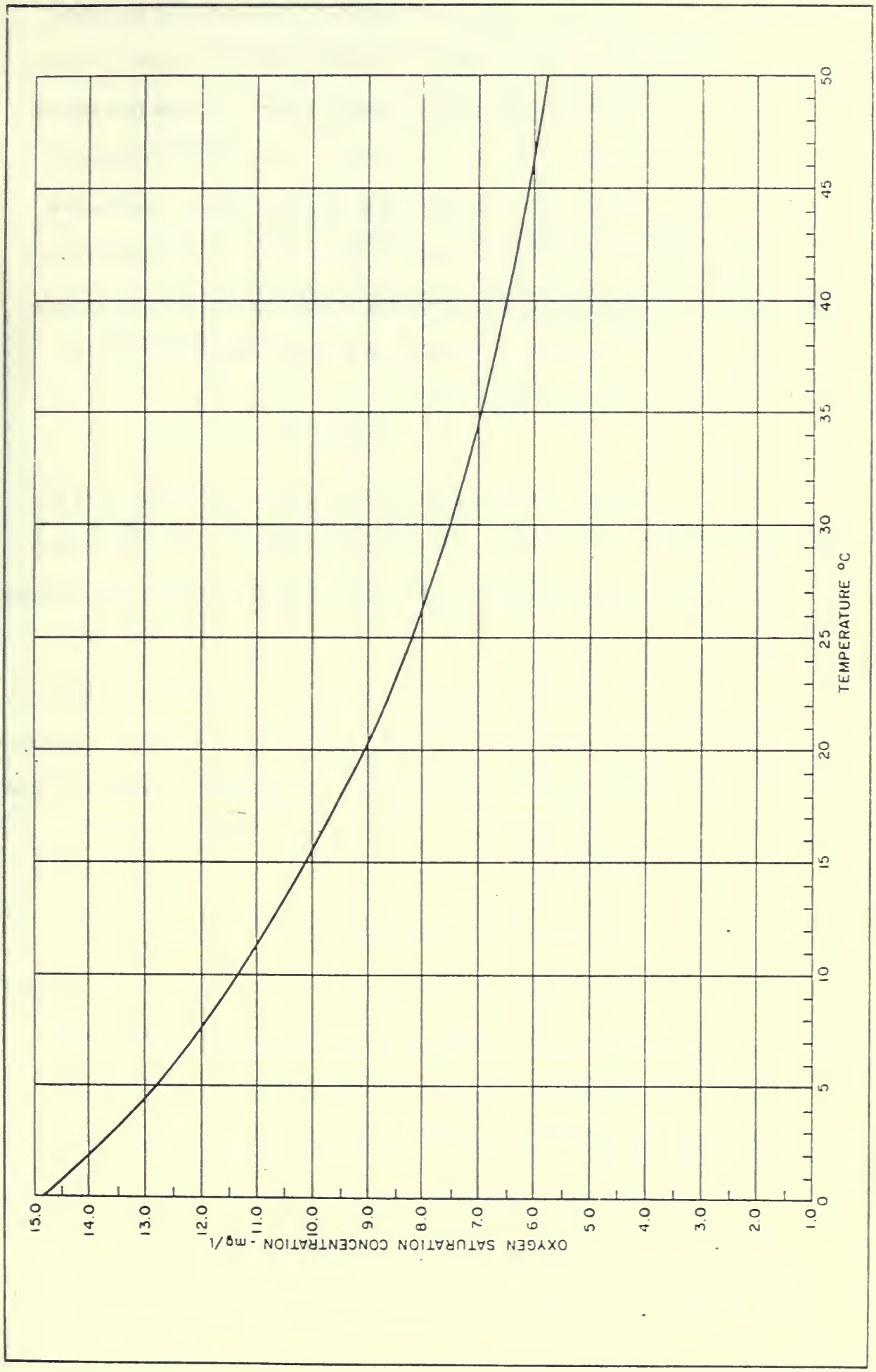


FIGURE 1.1 OXYGEN SATURATION - TEMPERATURE RELATIONSHIP

c. Freshwater Flow - provides not only dilution of the waste flow but also determines the velocity of the stream. The flow varies considerably from year to year. The low flow period usually occurs in summer-early fall and in conjuction with the still high temperature at that time of year, produce the most severe water quality conditions. Average annual flow and estimation of low flows can be obtained from an analysis of historical records and are usually expressed in cfs/sq. mi. or cfs units.

Stream flow data are published annually by the Ministry of the Environment in the Selected Streamflow Data for Ontario and by Environment Canada in the Surface Water Data, Ontario.

If the low flow data are not available, an estimate can be made by comparison with a nearby drainage basin of similar geophysical characteristics and adjusted to the drainage area of the stream in question.

The flow in the stream is obtained by multiplying the flow expressed in cubic feet per second per square mile of drainage area by the drainage area at the location of the waste water discharge. The total flow of the stream in cfs downstream from the point of discharge is therefore:

$$Q = f_5 DA + \text{waste Flow} \quad \ldots\ldots 1.5$$

where: f_5 = cubic feet per second per square mile

DA = drainage area (sq. mi.)

II PRELIMINARY MODEL OF STREAMS AND RIVERS

1. OUTLINE

Water quality analysis of streams may be classified in accordance with the reactive nature of the constituents in wastewater namely, conservative or non-conservative.

a. Conservative Substance

Conservative substances may include essentially non-disappearing substances, e.g. dissolved solids, chlorides, and nutrients (total nitrogen and total phosphorus).

The maximum concentration (C) of this type of constituent is at the point of discharge, assuming a completely mixed system, and is the mass rate of waste discharge (W) divided by the total flow (Q):

$$C = \frac{W}{Q} \quad \ldots\ldots 2.1$$

For multiple sources, the total concentration in the stream is the arithmetic addition of the individual effects plus the background value, Figure 2.1. For slowly decaying substances ($K^1 \leq 0.2$/day) it may be sufficiently accurate to add responses such as given by Equation 2.1 to indicate order of magnitude.

[1] *The reaction rate K, represents the rate which a substance builds up or die away and has units per time (T^{-1}). All K rates are to the base e.*

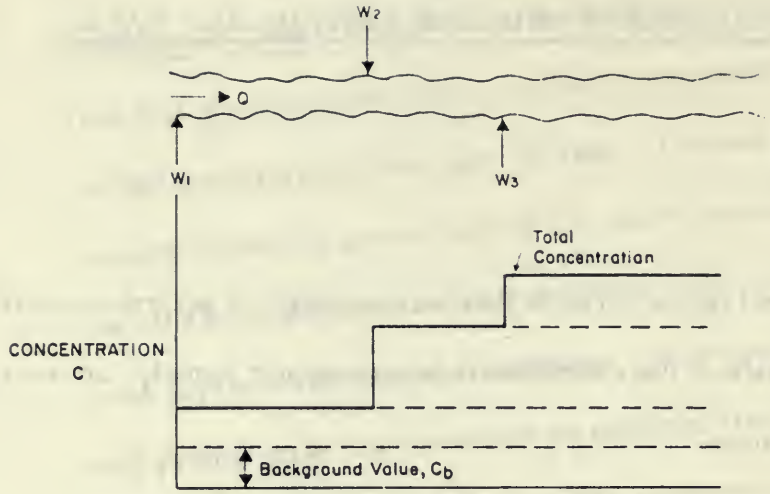


FIGURE 2.1 SUPERPOSITION OF MULTIPLE WASTE SOURCES OF CONSERVATIVE SUBSTANCES

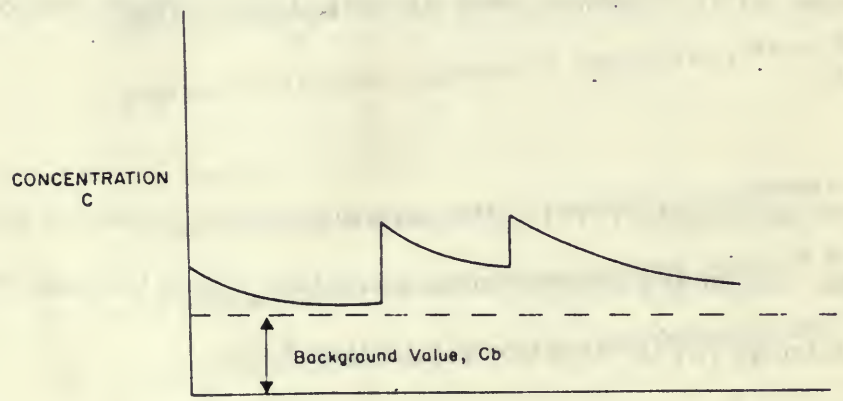


FIGURE 2.2 SUPERPOSITION OF MULTIPLE WASTE SOURCES OF NON-CONSERVATIVE SUBSTANCES

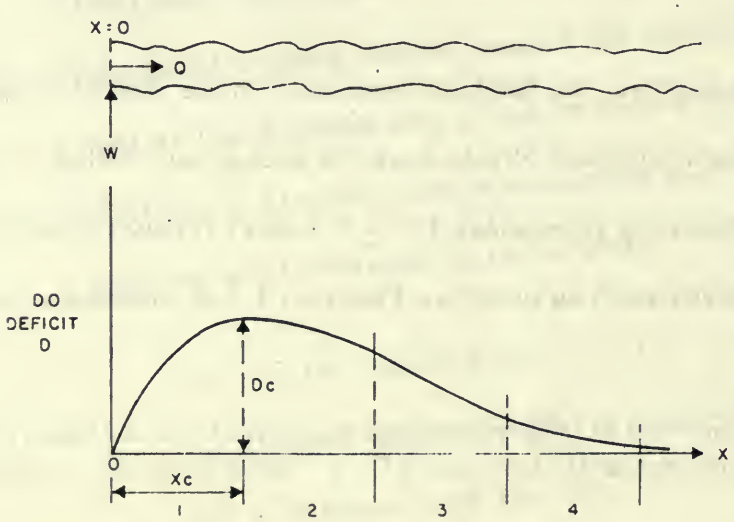


FIGURE 2.3 TYPICAL DO DEFICIT PROFILE

If this total concentration when compared to the water quality criterion is satisfactory, no further refinement is necessary. If the comparison is marginal it may be accepted since the analysis assumes a constant flow (Q) with no reactions taking place.

However, if the total concentration exceed the criterion it may be necessary to account for additonal inflow from groundwater or other sources entering along the length of the stream.

Should the concentration exceed the criterion subsequent to a detailed analysis of all sources of inflow to the stream, other methods would have to be considered for reducing the concentration further. These may include, reduction of the load at the source, flow augmentation or re-evaluation of the governing criterion.

b. Non-Conservative Substances

To account for substances which decay or undergo conversion by singular reaction e.g. bacteria concentration, radioactive matter, and oxygen demanding material, the following equation is adapted.

$$C = C_o e^{-\frac{K_x}{u}} \quad \ldots\ldots \quad 2.2$$

where:

C = Concentration at any location x

C_o = W/Q concentration in the stream at x=o

K = reaction coefficient (per day)

u = stream velocity

x = distance downstream

Ranges of values for reaction coefficients in freshwater streams for water temperature in the 20-25^{o}C range are:

Coliform Bacteria 1-3 per day

BOD 0.2-2.0 per day

Conversion to other temperatures can be made by:

$$K_T = K_{20} (1.047)^{T-20} \quad \ldots\ldots \quad 2.3$$

where, K_T is the reaction coefficient at temperature, T^{o}C and K_{20} is the reaction coefficient at 20^{o}C.

For multiple sources, the concentration profile is as shown in Figure 2.2.

c. Dissolved Oxygen Analysis

The concentration profile of dissolved oxygen downstream from a waste discharge is the result of a consecutive reaction. The first which is primarily the oxidative reaction of the residual organic matter, and the second which is the reaeration replacing the deficit caused by the first reaction. Consider a single source of waste water discharging at a rate of W into a stream with a fresh water flow of Q. The outfall is located at x=o downstream from which a typical dissolved oxygen deficit profile result, as shown in Figure 2.3.

The DO deficit (D) is given by the saturation value of oxygen (C_s), minus the dissolved oxygen (C) i.e.:

$$D = C_s - C \quad \ldots\ldots 2.4$$

The equation for the deficit profile is:

$$D = \frac{K_d L_o}{K_a - K_r} \left(e^{-K_r \frac{x}{\bar{u}}} - e^{-K_a \frac{x}{\bar{u}}}\right) + D_o e^{-K_a \frac{x}{\bar{u}}} \quad \ldots\ldots 2.5$$

where:

D = Dissolved oxygen deficit at a distance x from point of reference. (mg/l)

D_o = Initial dissolved oxygen deficit at point of reference. (mg/l)

u = Average stream velocity. (ft/day)

L_o = Ultimate BOD at point of reference. (mg/l)

Ka = Reaeration coefficient (day^{-1}) also expressed as K_2.

K_d = Deoxygenation coefficient. (day^{-1})

K_r = Coefficient of BOD removal. (day^{-1})

The Bod removal coefficient K_r, represent removal by deoxygenation and by settling, i.e. $K_r = K_d + K_s$. In many cases the settleable fraction is very small and can be neglected because the stream velocity prevents the organics from settling. Also depending on the waste source, e.g., effluent from an STP, the settleable organics would have been removed before the effluent enters the stream.

thus:

for $K_r = K_d$;

The magnitude and location of the maximum deficit, is given by:

$$D_c = \frac{K_d}{K_a} L_o e^{-\frac{K_d x_c}{u}} \quad \ldots\ldots 2.6$$

$$x_c = \frac{u}{K_a - K_d} \ln \frac{K_a}{K_d} \quad \ldots\ldots 2.7$$

where:

D_c = Maximum DO deficit. (mg/l)

and;

x_c = Distance to location of maximum deficit from point of reference. (ft.)

Substituting equation 2.7 into 2.6 and rearranging terms:

$$\frac{D_c}{L_o} = \phi^{\frac{\phi}{1-\phi}} \quad \ldots\ldots 2.8$$

where:

$$\phi = \frac{K_a}{K_d}$$

The dimensionless ratio ϕ is known as the assimilation ration. A practical range of ϕ is from 0.1 to 20 for which D_c/L_c values are shown in Figure 2.4.

Equation 2.8 forms the basis for the dissolved oxygen response in streams due to a single waste source. This will be discussed further together with a detailed description of reaction coefficients and assimilation ratio ϕ.

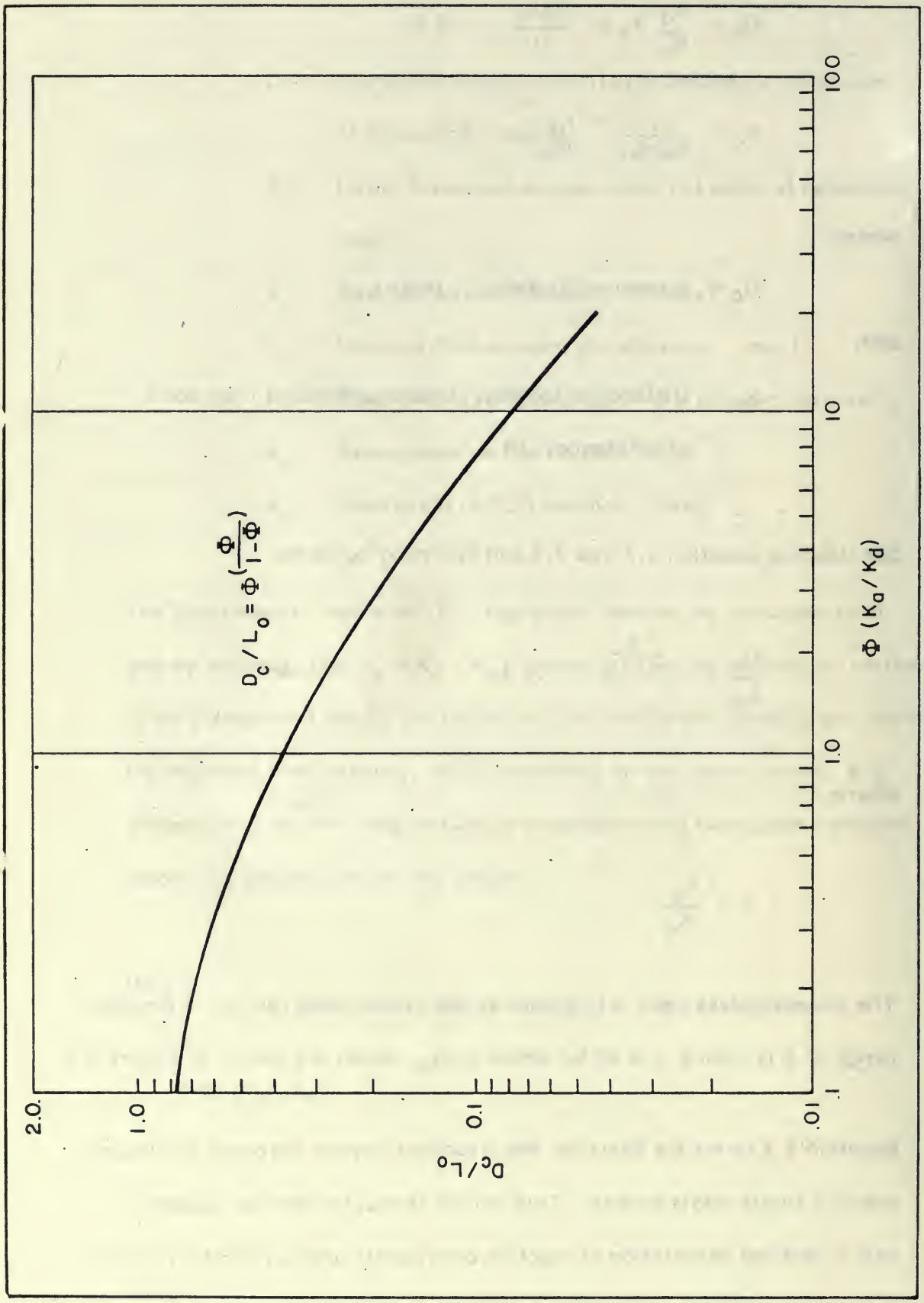


FIGURE 2.4 RATIO D_c / L_o AS A FUNCTION OF ASSIMILATION RATIO, Φ

2. Single Waste Source - Graphical Dissolved Oxygen Analysis

The estimation of the necessary treatment required to meet dissolved oxygen criterion for a single waste source in a stream can be summarized in a nomograph as given by chart A. This chart can be used to test the efficiency of several treatment levels in meeting dissolved oxygen criterion. The procedure can be reversed to estimate treatment efficiency required for a given minimum dissolved oxygen crtierion.

Figure A-1 represents the discrete levels of treatment considered. The solid lines represent the percentage removals shown in Table II. The broken lines represent removal efficiencies greater or less than those in Table II to allow for variable plant performances. Figure A-1 therefore permits estimation of the total oxygen demand in pounds/day that would be discharged from the given design population and treatment level.

Figure A-2 requires the total streamflow which includes the streamflow above the discharges plus the flow for the waste source itself, (Equation 1.5). Given the total oxygen demand from Figure A-1 and the streamflow, Figure A-2 is used to estimate the initial concentration of the total oxygen demand in mg/l resulting after dilution in the stream, (Equation 2.1). A zero upstream BOD concentration is assumed which will be accounted for in Figure A-4.

Figure A-3 incorporates the fundamental reaeration capacity of the stream using several different data levels. Information on the stream can be at one or all of the following levels:

a) Qualitative description, e.g., shallow, main drainage rivers, impounded rivers, etc.,

b) A measurement of depth of the river, a key variable,

c) Estimate of δ, the ratio of the reaeration coefficient, K_a and the deoxygenation coefficient, K_d.

Given the previous description of the stream, the maximum DO deficit (mg/l) using Figure A-3 can be estimated. Note that a zero initial DO deficit has been assumed. Any "background" DO deficit is incorporated at the conclusion of the analysis. Figure A-3 is constructed from Equation 2.5, a discussion of which is given in the appendix.

Figure A-4 incorporates the variation of the DO saturation level with maximum water temperature (see Figure 1.1). With the maximum DO deficit and maximum water temperature, the minimum dissolved oxygen (mg/l) can be estimated. The "background" deficit is included at this point. In the absence of any data, it is recommended that the DO background deficit of 1.0 mg/l be considered as a minimum level.

A constant background DO deficit is incorporated for two reasons

a) computational simplicity and,

b) existance of other sources and sinks of dissolved oxygen.

The theory indicates that an initial DO deficit decays exponentially at a rate given by the reaeration rate and stream velocity. To allow for the existence of other phenomena which affect DO, including benthal oxygen demanding material, algal photosynthesis and respiration, incremental additions of oxygen demanding loads from agricultural and urban drainage, among others, it was desirable to incorporate all these effects in a single constant background DO deficit.

The procedure described above provides a first estimate of the minimum dissolved oxygen which is then compared to the set criterion. If the dissolved oxygen for the given level of treatment is less than the criterion, the procedure can be reversed by starting at the required DO and continuing through Figures A-4, A-3, and A-2 using the same water temperature, stream reaeration capcity and total streamflow. One now enters Figure A-1 with the required total oxygen demand mass discharge. For the fixed design population, the next highest level of treatment is chosen which will equate or exceed the required pounds/day discharge.

Example 1:

A conventional secondary sewage treatment plant with phosphorus removal to serve an ultimate population of 150,000 is to be located at a point X on river Y. What will be the minimum dissolved oxygen level measured downstream as a result of the treated waste discharge?

Procedure:

a) Basic information required:

1. Plant size - the town is typical of Southern Ontario municipalities with a good mix of industry, commerce, apartment dwellings and individual houses. Per capita sewage flow selected is 150 gallons per day.

2. Low Flow - river Y has a continuous recording flow gauge near its mouth. The summer low flow (7Q20) is calculated and the results extrapolated on an drainage area ratio to point X. Therefore, calculated low flow is 85 cfs.

3. From water quality monitoring records, the maximum water temperature is 25°C and the background dissolved oxygen deficit in mid-summer is 1 mg/l.

4. A field trip was taken and the river was observed from several vantage point downstream from point X. Based on field observation, a mean depth of 7 feet was selected.

Summary of Basic Information

Design Population	-	150,000 persons
Treatment type	-	Secondary
Per capita waste flow	-	150 gallons/day
Calculated drought flow	-	85 cfs
Maximum water temperature	-	25°C
Mean depth of river	-	7 feet
Upstream DO deficit	-	1.0 mg/l

Total flow calculation:

River flow = 85

River flow = $150 \times 150{,}000 \times 1{,}86 \times 10^{-6}$ = 42

Total = 127 cfs

b) Graphical solution: (see Figure 2.5)

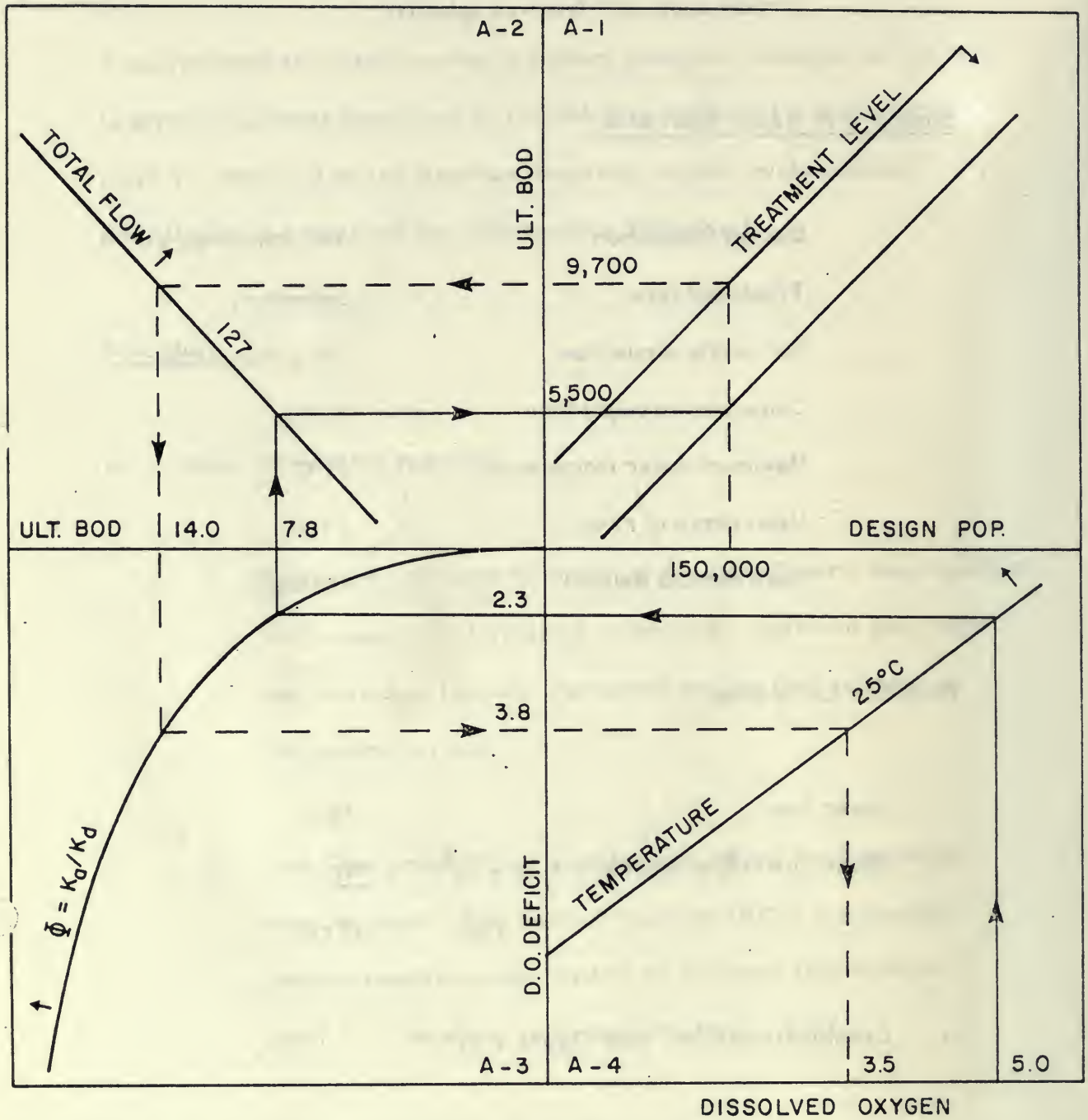


FIGURE 2.5 EXAMPLE 1 AND 2.

Enter Chart at:

Figure A-1, design population of 150,000 with secondary treatment gives an effluent load of 9,700 lb/day ultimate BOD.

Figure A-2, drought flow of 127 cfs and effluent load of 9,700 lb/day ultimate BOD gives upstream oxygen demand concentration of 14 mg/l.

Figure A-3, for an intermediate channel (7 feet, approximately) maximum DO deficit is 3.8 mg/l.

Figure A-4, at a temperature of 25°C, assuming a background deficit of 1.0 mg/l gives a minimum dissolved oxygen of 3.5 mg/l.

Example 2:

Given the same information as in example 1, determine the waste loading and treatment level required to maintain a dissolved oxygen level of 5.0 mg/l in the river.

Procedure:

Enter Chart at:

Figure A-4, for a minimum DO of 5.0 mg/l with a 1.0 mg/l background deficit and 25°C, the, maximum allowable deficit is 2.3 mg/l.

Figure A-3, for an intermediate channel, the allowable instream ultimate BOD concentration is 7.8 mg/l.

Figure A-2, for a drought flow of 127 cfs, the allowable treated discharge is 5,500 lb/day ultimate BOD.

Figure A-1, for a 5,500 lb/day discharge and a design population of 150,000 the treatment level will have to be bordering the advance level to achieve the 5.0 mg/l minimum DO in the stream.

Example 3:

Estimate the streamflow necessary for discharging a seasonal lagoon while maintaining a minimum DO of 5.0 mg/l, given the following information:

Population	1000
Per capita flow	100 gals.
Retention time	180 days
Lagoon size	13.3 ac. by 5 feet depth
Lagoon effluent	25 mg/l BOD_5, $k = 0.25$
Stream	Upstream feeder, depth approx. 4 feet

Procedure:

Sewage flow	1000 x 100 = 100,000 gals/day
	= 0.186 cfs

For a continuous discharge over three days, flow rate is:

$$\frac{13.3 \times 43560 \times 5}{3 \times 24 \times 3600} = 11.2 \text{ cfs}$$

Waste Load:

$$\text{UBOD} = \frac{\text{BOD}_5}{(1 - e^{-kt})}$$

$$\text{UBOD} = \frac{25}{(1 - e^{-0.25 \times 5})} = 35 \text{ mg/l}$$

thus waste load = $11.2 \times 5.4 \times 35 = 2120$ lb/day

Enter Chart at:

Figure A-4, for minimum DO of 5.0 mg/l with a 1.0 mg/l background deficit and at 25°C, the maximum allowable deficit is 2.3 mg/l.

Figure A-3, for an upstream feeder, the allowable instream ultimate BOD concentration is 13.0 mg/l.

Figure A-2, for a waste load of 1815 lb/day and allowable BOD concentration of 13.0 mg/l, the total flow is 31 cfs.

Similarly :

Days	cfs -	Load Lb/Day	Total Flow	Stream Flow
3	11.2	2120	31	19.8
5	6.2	1175	15	8.8
7	4.8	910	12	7.2
10	3.4	645	9.4	6.0

The rate of discharge and duration can then be selected based on the streamflow. By examining past flow records, a suitable period can be selected for partial or complete discharge of the lagoon.

NOTES

APPENDIX

REACTION COEFFICIENTS

1. Aeration Coefficient

The transfer of oxygen from the atmosphere to water is essentially a surface controlled phenomenon. The relationship between the surface transfer coefficient K_L and the volumetric coefficient K_a is:

$$K_a = K_L \frac{A}{V} = \frac{K_L}{H} \quad \ldots\ldots \text{A-1}$$

where: A = surface area of river reach

V = volume at river reach

H = average depth

It has been shown that these coefficients are directly proportional to the velocity of the stream, raised to some power "a" and inversely proportional to the average depth of the stream to a power "b" namely, $\left(\frac{D_L V}{H^{3/2}}\right)^{\frac{1}{2}}$.
A plot of the transfer coefficient verses the average depth of the average depth of the stream is shown in Figure 1 for various velocity ranges. Figure II is a plot of K_a (evaluated from Equation A-1), verses average depth for three velocity ranges. The three curves are indicative of the range of water surface conditons, and stream bed characteristics.

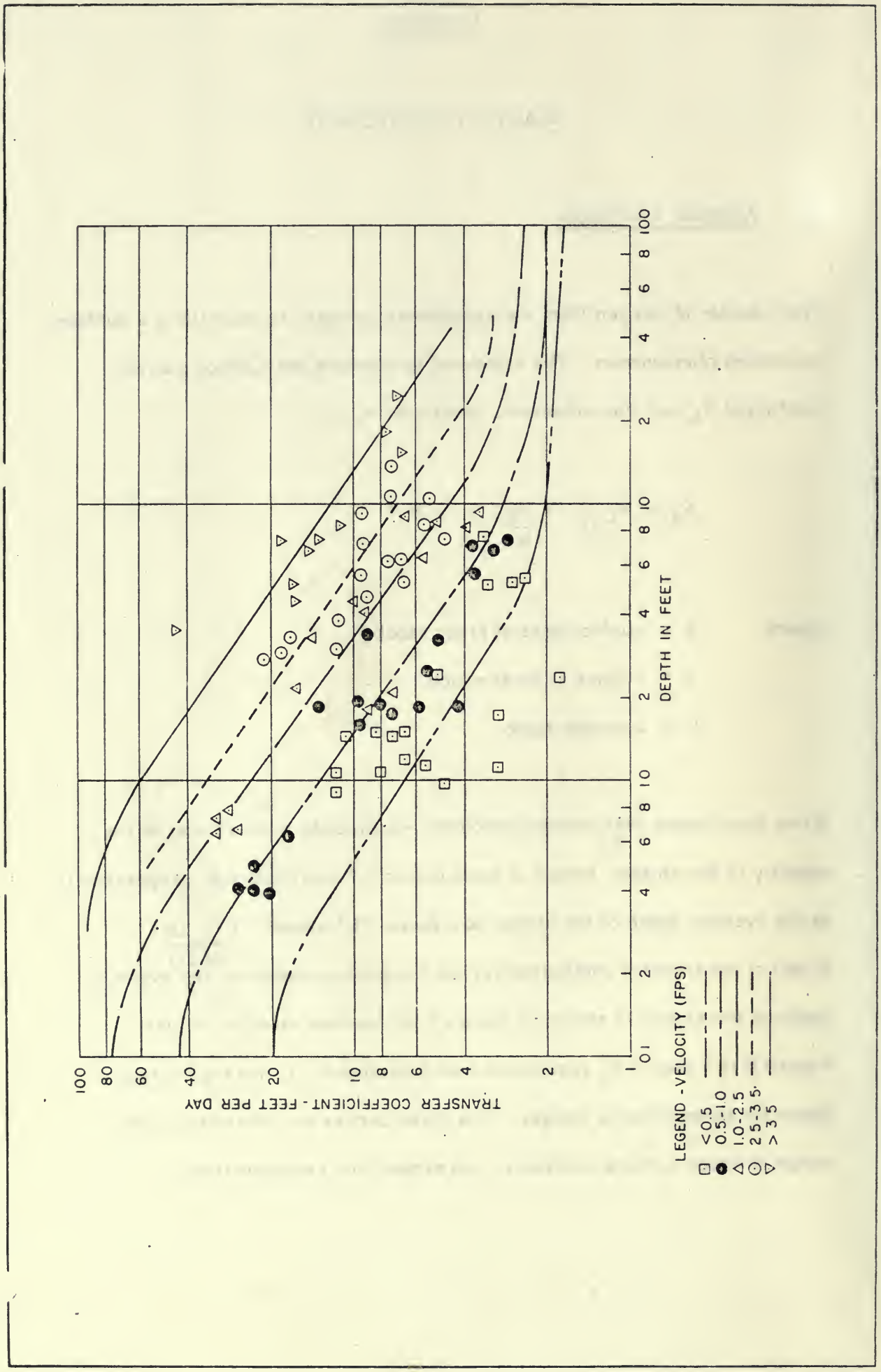


FIGURE I. TRANSFER COEFFICIENT (KL) AS A FUNCTION OF DEPTH
(From Hydroscience Inc. 1971)

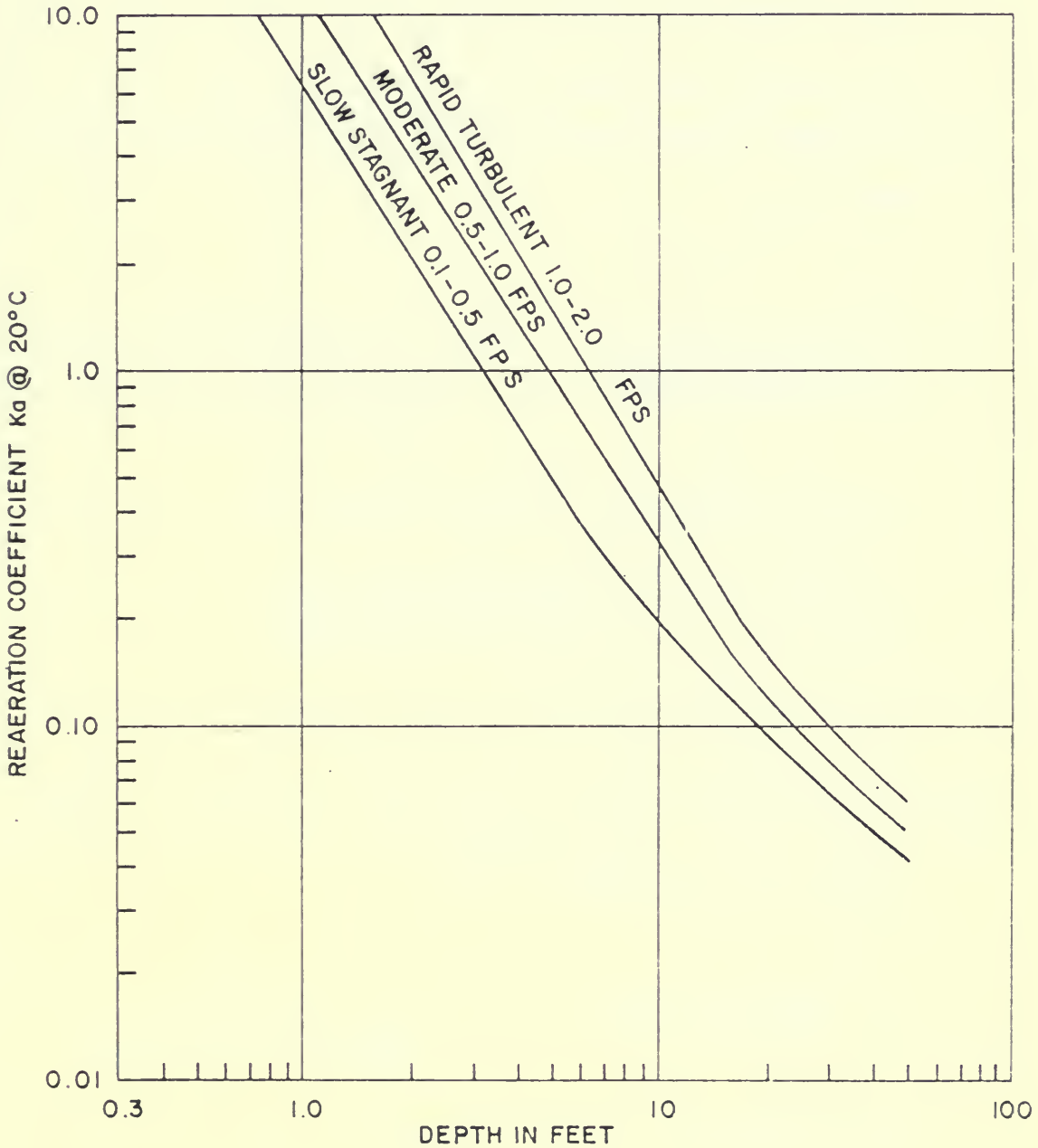


FIGURE II. REAERATION COEFFICIENT (Ka) AS A FUNCTION OF DEPTH
(From Hydroscience Inc. 1971)

2. Deaeration Coefficient

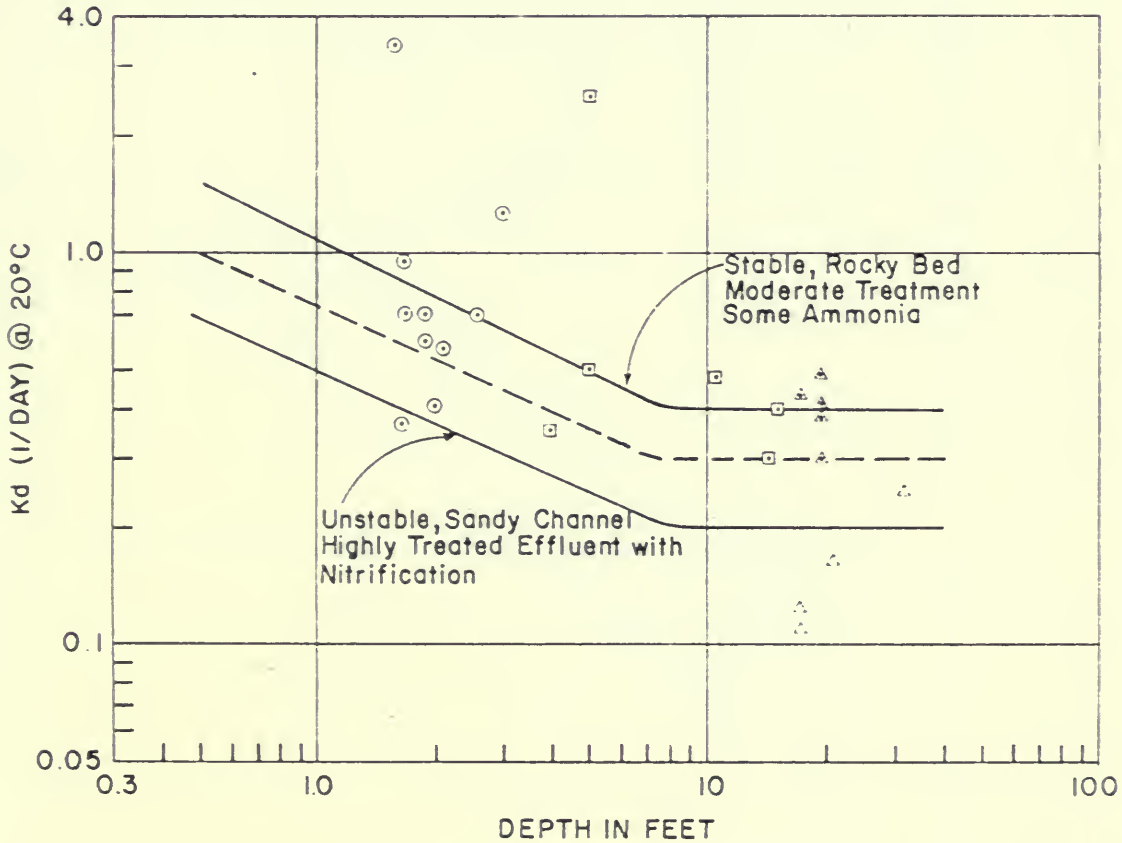
A similar correlation has been developed for the deaeration or deoxygenation coefficient K_d and average depth H. This correlation is based on the fact that the greater the wetted perimeter to cross-section, which is equivalent to small depth, the greater the contact with the biological forms on the stream bed. Figure III is a plot of K_d verses average depth based on data surveys.

The upper line represents a stable, rocky bed with benthal communities, whilst the lower line represent a unstable bed with sparce benthal communities.

In addition, the nature of the residual organic matter in the effluent from the treatment plant is a determining factor. The lower limit represents highly treated, well oxidized effluents with efficient secondary sedimentation of the active bacterial populations. The residues are therefore highly stable organically with low rates of oxidation. The upper limit is representative of waste with more residual organic matter, some of which is capable of relatively rapid oxidation.

3. Assimilation Ratio

The assimilation ratio = (K_a/K_d) is plotted in Figure IV verses the average depth, the correlation being developed directly from those in Figures II and III. The heavy solid line represents the average case. If actual



LEGEND

⊙ Shallow Streams (1 - 3 Ft.)
⊡ Medium Streams (3 - 15 Ft.)
△ Deep Rivers (> 15 Ft.)

FIGURE III. DEOXYGENATION COEFFICIENT (Kd) AS A FUNCTION OF DEPTH
(From Hydroscience Inc. 1971)

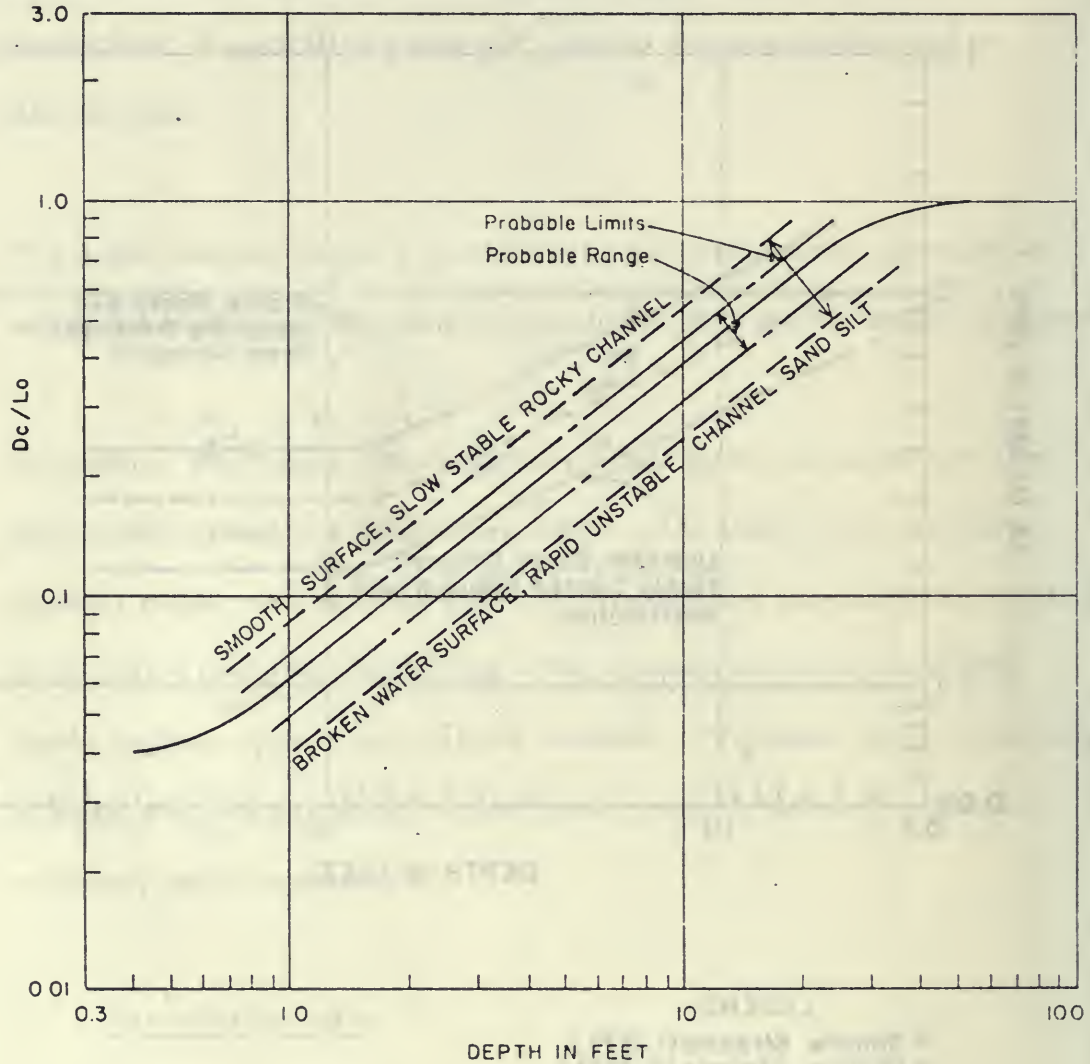


FIGURE V. RATIO Dc/Lo AS A FUNCTION OF DEPTH
(From Hydroscience Inc. 1971)

data are available from stream surveys to determine k_2, this information should obviously be employed in the BOD analysis. If no data are available to estimate the individual coefficients, the ratio may be estimated simply from the depth. Further information on the nature of the stream and the anticipated treatment enables further refinement within the range indicated.

The final step is the conversion of the ratio ϕ, to (D_c/L_o), indicated in Figure 3 of the main text. The relation between D_c/L_o and the average depth, H, with limits as indicated, is shown in Figure V.

The relation presented in Figure V, is the basis of the nomographs presented in the stream analysis section.

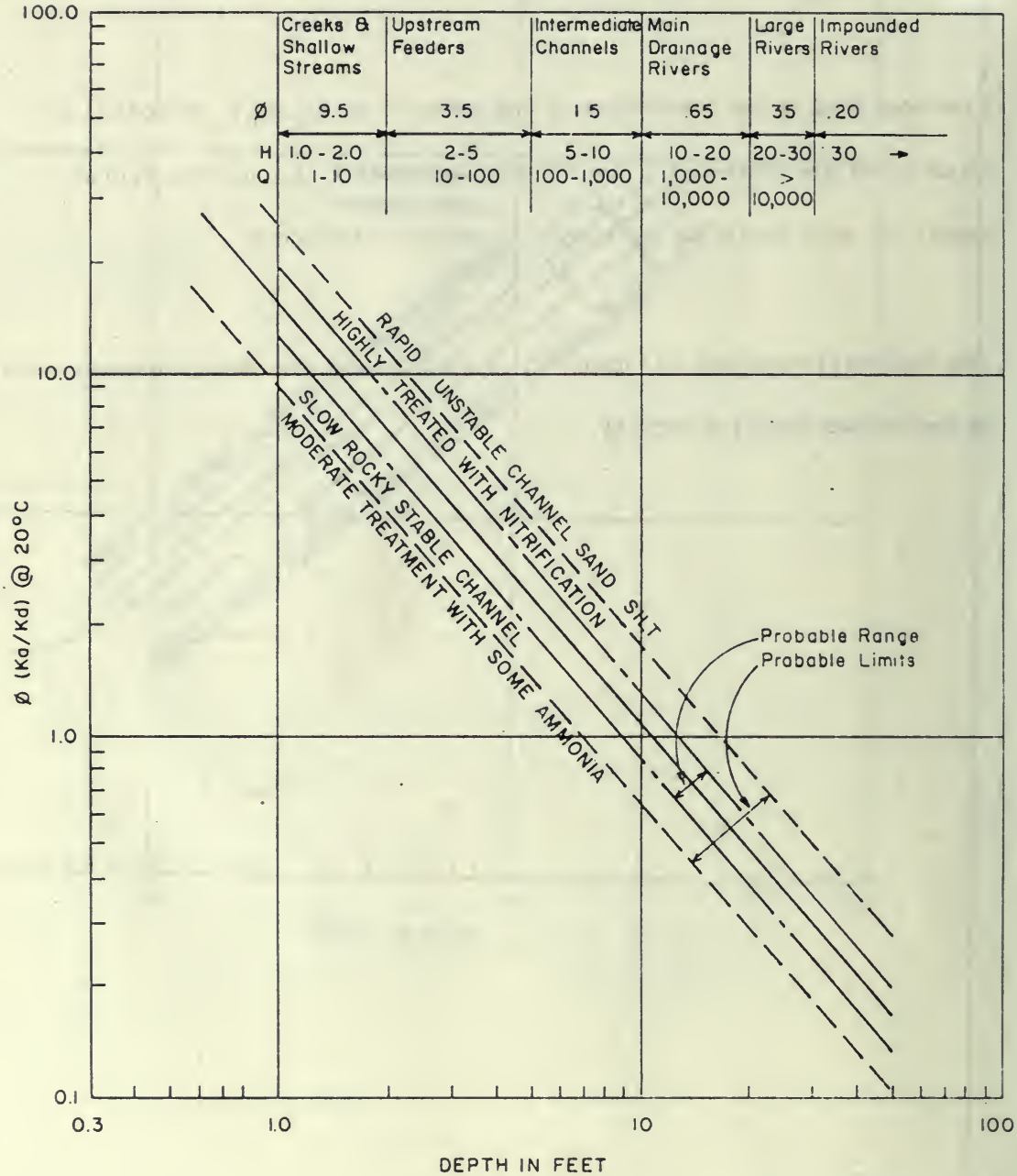


FIGURE IV. ASSIMILATION RATIO (Ø) AS A FUNCTION OF DEPTH
(From Hydroscience Inc. 1971)